AIRPORT ENGINEERING

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By H. Oakley Sharp, G. Reed Shaw, and John A. Dunlop

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A modern treatise on airport construction materials, design of buildings, pavements, and other design and construction details. *148 pages · 253 illustrations ·* 8½ by 11

Airport Engineering

- H. Oakley Sharp

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PREFACE

Air transportation in peace and in war has reached the point in its progress where special engineering considera-tion must be given the ground facilities which serve as the connecting link between surface and air transportation.

Rail and water transportation depend upon the terminal facilities offered at rail centers and ports for the successful transfer of passengers and goods. Airport engineering must keep pace with the development of planes and the rapid increase in the use of the airways. Consequently civil engineers and city planning officials must give special attention to the air terminals which will be provided to serve this increasingly important means of transportation.

The engineer must make a special study of the airport terminal in order to meet the flight requirements of the planes to be used; he must plan an arrangement of service buildings which will guarantee a steady flow of traffic where land and air lines meet.

The grading, paving and drainage problems encountered on an area being prepared for a landing area are in many ways different from those with which the engineer has had to deal in connection with highway construction. They result from the magnitude of the landing areas, provision for year-round operation of the landing fields, and the services which must be rendered to permit the safe and comfortable operation of regular schedules for freight, mail and passengers. The fundamentals of grading and drainage are presented as they particularly apply to air-port construction.

The authors of this book have endeavored to present the major problems which must be studied by the engineer engaged in airport construction. The basic aspects of planning the site and the relation of the location of the site to the regional plan are presented as considerations preliminary to airport construction.

The importance of stabilization to airport construction cannot be overemphasized. Chapter VII is devoted to this subject with particular emphasis on its applications to the manipulation and treatment of soils for immediate and future heavy loads.

The paved runways of an airport are basically highways along which the planes travel while on the ground. The loads encountered on airport runways differ in many respects from those on highways. With rapidly changing designs of planes these loads change materially. Sufficient information is now available to indicate the differences in the concentration, impact, and repetition of the loads from those of highway loads. The basic methods of design for subgrade support and surface pavements are presented in Chapters VIII and IX. It is anticipated that these basic principles will continue to be sound design even with the greater magnitude of load encountered in the plane of the future.

The selection of a surface for the runways, taxiways, aprons, and landing strips is a matter of considerable importance and there are differences of opinion as to which kind makes the best surface. The authors have attempted to discuss all the different types from the view-point of design, construction, and use.

Although the engineering aspects of airport construction do not involve many design procedures which are solely adaptable to airports it has been deemed advisable to gather information and data concerning this new problem in the engineering field. It is therefore the objective of the authors to present as complete a picture of the engi-neering work involved in the planning and design of a modern air terminal as can be adequately covered in a textbook designed for undergraduate civil engineering students.

Many of the fundamental theories are developed in this book; diagrams and illustrations form an important feature for instruction purposes.

Acknowledgment is made of the assistance and material furnished by the Portland Cement Association, the Asphalt Institute, the Armco Drainage Products Association and the Concrete Pipe Association. Several men connected with these organizations deserve particular mention as they have been of great assistance to the authors: Mr. L. E. Andrews, Mr. W. B. Kane, Mr. Bernard Gray, and Mr. H. E. Cotton deserve special mention.

Acknowledgment is also made of the many fine photo-graphs from the Caterpillar Tractor Company, the Irving Subway Grating Company, the Advertising Producers

Associated, the Hi-Way Service Corporation, the Portland Cement Association, the Asphalt Institute, the Armco Drainage Products Association, and the Public Roads Administration.

The publications of the Civil Aeronautics Administration have been consulted frequently and information from these publications has been used throughout this book. The work of that organization forms the bases for most of the standard specifications on airports and acknowledgment is made here and throughout the book for the assistance of this government agency.

H. O. S.
G. R. S.
J. A. D.

RENSSELAER POLYTECHNIC INSTITUTE
July, 1944

CONTENTS

Chapter I

Introduction

1. Engineering Importance. The building of airports, both at home and abroad, is one of our most important present engineering activities. The necessity for constructing these airports at a rapid rate, under the control of various organizations, has resulted in different methods of approach to the design which reflect the experience of each group in charge. Development in the size and number of planes has been so rapid that layouts which were satisfactory a few years ago are now almost obsolete. When the huge planes now under discussion come into actual use, it will be necessary to build new landing fields of proper size to meet the take-off requirements of large and heavy planes.

2. Looking into the Future. The number of airports constructed under accelerated Army and Navy programs is large but these programs will not provide an adequate number of airports for the future demands of the civilian population during peacetime. Commercial flying and private flying will both increase at astonishing rates. Thousands of airports and flight strips will spring up, not only in the United States but also all over the world. The hundreds of thousands of young men from all parts of the country who are now being trained to fly for the various branches of the service will not want to be "grounded" when they return home. Even though they return to the farms, they are certain to be eager purchasers of the "flivver" plane which will be available. Likewise, business men who have recently been obliged to fly because of war necessity have learned to like it and never again will be content to spend 36 hours on a train for a journey that can be made in 5 or 6 hours by air.

Plans have been made and construction started for the Idlewild Airport on Jamaica Bay in New York. This is a huge undertaking and will cover an area of approximately 3 by 2 miles with room for expansion. It will be six times larger than LaGuardia Field and will cost approximately $100,000,000. The drainage system alone will cost $1,500,000. There will be six miles of hangars for land planes and one mile for sea planes. Thirteen and one-half miles of runways will provide a capacity for handling 900 planes in 15 hours. Future plans include 100-passenger planes which will furnish the transportation to European countries on an hourly schedule.

3. The Engineering Problem. Civil engineers, both young and old, should be giving careful attention to this huge new business of flying, and to what it means in the use of construction materials, design of buildings, design of pavements, and all other pertinent details.

Many engineers who are designing and building airports are, unconsciously, perhaps, thinking only in terms of highways without fully appreciating the many points of dissimilarity. The difference in character of loadings, rates of load repetition, and operational details allows experience in highways to carry only so far, for beyond that point a new field is opened and there is yet lacking much of the data which will be required to predict accurately service behavior under conditions of the future.

4. All-Over Landing Fields. The all-over field in which the entire area may be used for landing and taking off provides the most desirable field and furnishes the most available landing area for given over-all dimensions. For this type of field the entire area must be properly graded, drained, and surfaced. Large numbers of planes may take off or land in a short period of time as the planes may follow parallel lines, a reasonable distance apart, in any direction. This direction is determined by the existing wind condition and it is possible to provide for every wind direction. In an emergency large numbers of planes can take off or land at the same time; this permits formation take-offs and landings which are necessary for military operations. This type is as bombproof as any air field can be because any part of it may be used for landing and taking off.

If the soil has high supporting power and the natural

drainage is adequate it may be necessary only to treat the surface to prevent excessive dust for this type of field. The surface usually used is turf with paved aprons adjacent to the hangars and paved surfaces only at points of concentrated traffic. In Europe, where this type of field has been the most popular, many airports have been paved over the entire area. The soil condition is the major factor in deciding whether or not the all-over field can be used and, if the soil is satisfactory, a suitable drainage system must be provided to permit the field to be used during periods of continued wet weather.

Another problem encountered is that of snow removal. The usual solution is to clear the snow along strips to form runways; at such times the field is not used as an all-over field.

5. Landing Strips or Runways. The drainage and surfacing problems encountered in providing an all-over field have led to the building of fields having definite runways to be used for landing and taking off. In most instances this arrangement simplifies the drainage problems and makes it possible to provide a suitable surface on only a portion of the field upon which all the traffic will be concentrated. This type of field with paved runways is usable under the most adverse weather conditions. When the soil conditions are not satisfactory a pavement is used to distribute the load over a larger area of the soil and at the same time to prevent dust and keep the water from entering the subsoil.

For military airports the runway type of field can be seriously damaged by bombing. Usually there are intersections of the runways which, if destroyed, will put the airport out of operation.

This type of field has been the most popular in the United States and most of the commercial fields are of this type.

6. Flight Strips. Flight strips are being designed for some parts of the United States and federal appropriations have been made to finance their construction. They consist of long straight sections of highway pavement laid adjacent to existing highways. The widths are usually wider than the highway pavement and are constructed so that they may be used for landing strips or for vehicular traffic. Often they are built on the existing right of way and follow the same grade line as the highway; this simplifies the earthwork and drainage problem.

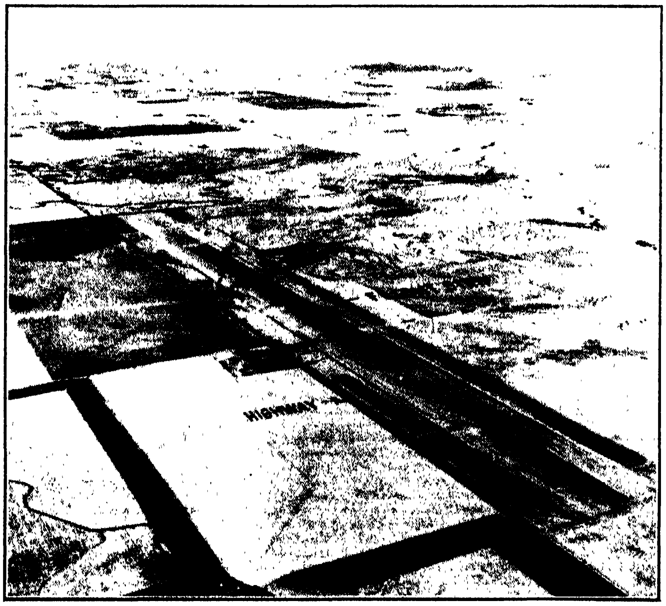
The purposes of a flight strip are:

1. To provide auxiliary landing fields for military purposes and to assist in the dispersal of planes from nearby airports.
2. To provide landing facilities for civilian flyers.
3. To provide basic landing facilities for small cities or groups of towns for air feeder service and air cargo.
4. To provide auxiliary landing facilities for all types of aircraft.

The Commissioner of Public Roads of the United States was authorized by the Federal Highway Act of 1940 to study the possibilities of constructing flight strips adjacent to highways in various parts of the country. Studies were conducted in conjunction with state highway departments and authorization for the construction of flight strips was approved in the National Highway Act of 1941 which states:

> *Sec. 8.* FLIGHT STRIPS. In order to insure greater safety for traffic on the public highways by providing additional facilities in connection therewith to be available for the landing and take-off of aircraft, the Commissioner of Public Roads is authorized to provide, in cooperation with the Army Air Corps, for studies and for the construction of "Flight Strips" adjacent to public highways or roadside development areas along such highways. The acquisition of new or additional lands necessary for such projects may, to the extent determined by the Federal Works Administrator, be included as part of the construction thereof and Federal funds shall be available to pay the cost of such acquisition. For carrying out the purposes of this section, there is hereby authorized to be appropriated during the continuance of the emergency declared by the President on May 27, 1941, in addition to any funds that may be available under any other appropriation, the sum of $10,000,000, which shall be available, without regard to apportionment among the several States, for paying all or any part of the cost of such projects.

Subsequently, two appropriations for flight strips have been made, each in the amount of $5,000,000.



Courtesy Public Roads Administration

FIG. 1. Bituminous-surfaced runway for flight strip showing adjacent highway. Runways are also constructed of concrete.

7. Sea Plane Bases. Many airports are built near the coast line or near large bodies of water. Sea planes take off and land on water but some provision must be made to

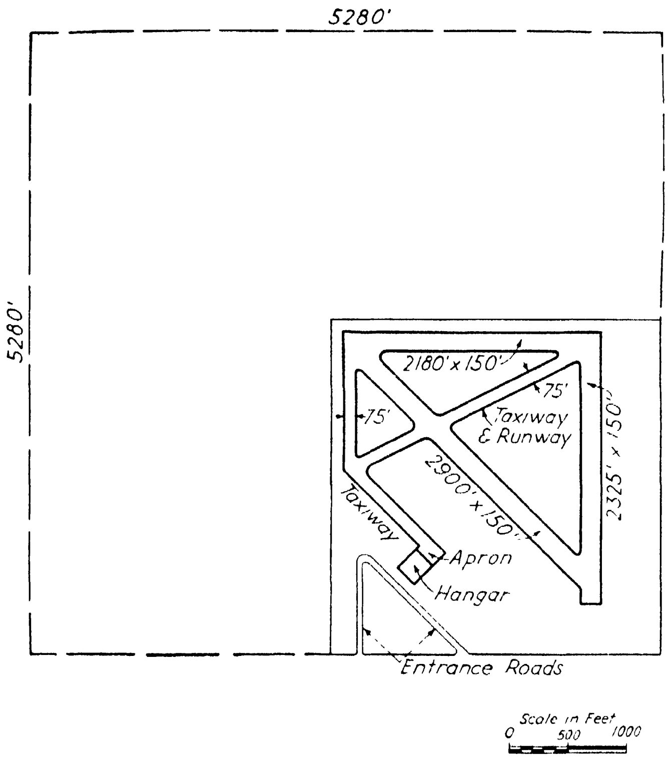


Fig. 2.

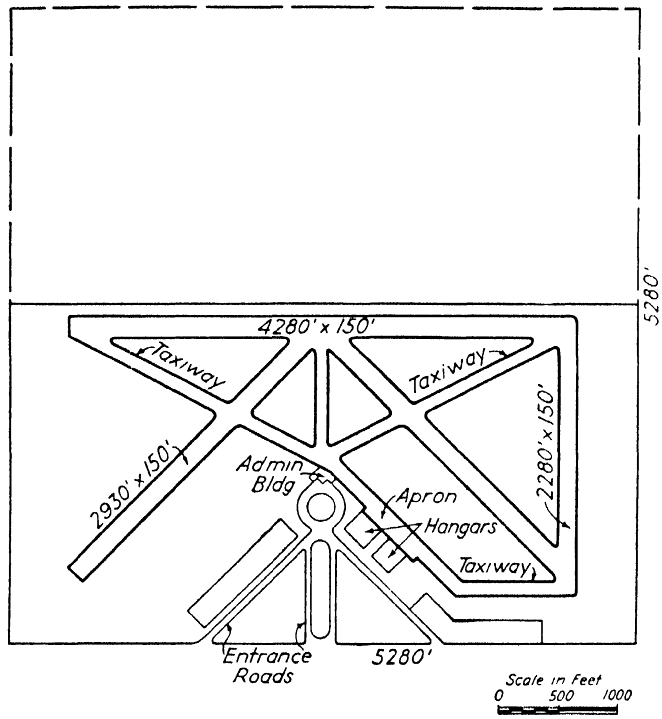


Fig. 3.

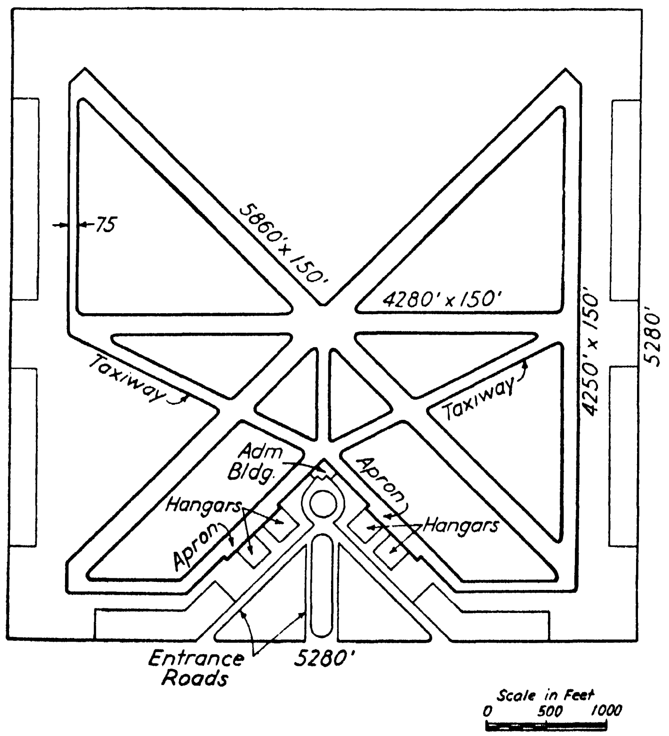


Fig. 4.

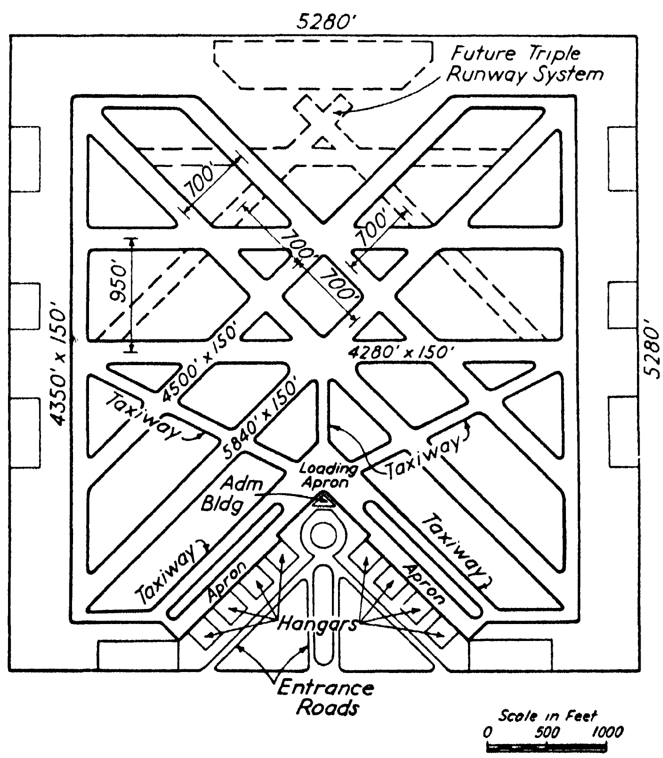


Fig. 5.

move them from water to land. This is usually accomplished by ramps on which are constructed small marine railways to facilitate the movements of large planes on which no wheels are placed. Some bases are constructed for sea planes only but most coastal bases are built with facilities for both land planes and sea planes.

8. Development. The development of an airport may cover a period of years but the plans should be laid so that there will be ample room for expansion. The amount of construction done at one time will depend upon the available funds and the current traffic. However, the engineer should think of the future and make the necessary plans for additions to the airport. The series of developments shown in Figs. 2, 3, 4, and 5 will serve as an illustration of the type of planning necessary.

Figure 2 is a simple airport which may be constructed at moderate cost but the land for future development is made available at the time of the original purchase. The maximum length of runway in this design is 2900 feet, which is not long enough for heavy planes. This may be expanded as traffic develops and longer runways may be provided as shown in Fig. 3. Greater lengths and more runways are provided on this same development as shown in Fig. 4. Finally, as traffic becomes heavy the expansion shown in Fig. 5 becomes necessary.

It will be noted that on the original plan (Fig. 2) an ample area was provided for present use and future expansion; the engineer must have in mind the ultimate needs when he starts the first simple airport design.

Chapter II

Size and Site of an Airport

9. Importance of Location Planning. The planning of a new location for an air terminal should embrace all the factors of regional planning and, in addition, the position of the proposed air terminal in the national network should be carefully analyzed. There will be more than one site to which consideration must be given and the economical use of funds will depend upon the scope and vision employed in the preliminary planning.

Many railroad failures were due to an inadequate reconnaissance and too hasty construction over lines which later proved to be too costly to operate when competing roads were built over more favorable lines. One of the major problems in air terminal design and construction will be the utilization of existing airports which have grown from the barnstorming days of aviation. The total absence of planning and the inadequacy of many of these existing ports will preclude the further expansion of such sites to provide the facilities required of air terminals today. The engineer should not hesitate to scrap an existing air field when it is apparent that the limitations of the site are such as to handicap the expansion and utilization of the area to the end that an efficient and convenient air terminal may serve the region.

10. Classification of Airports. There are two general classes of airports: (1) military airports, which include all airports used by the Army, Navy, and Marines, and (2) civilian airports, which include all airports for commercial and private use. In many instances civilian airports have been taken over for military use and it is quite likely that many military airports which have been constructed will eventually be used for civilian air transportation. Most of the specifications set up by the various military and civilian units were molded from those established by the Civil Aeronautics Administration, so a change from one type to the other does not involve many major changes.

11. Military Airports. Military airports are classified into three groups: (1) single-engine schools, (2) multi-engine schools, and (3) bomber and air depots.

TABLE I

Type of Field	Wheel Loads (lb.)	Tire Pressure (lb./sq. in.)	Contact Area (sq. in.)
Single-engine schools	15,000	55	273
Multi-engine schools, tactical stations other than bomber, technical schools, etc.	37,000	65	570
Bomber and air depot fields	60,000	75	800

12. Commercial Airports. Commercial airports have been divided into five classes by the Civil Aeronautics Administration. A table compiled from information published by the Civil Aeronautics Administration through its Technical Development Division gives a summary of these classifications and the requirements which are considered essential for safe operation in each group. It is given on page 6 as Table II. This table shows the lengths of landing strips necessary for each type of traffic.

13. Analysis of Traffic To Be Served. The first decision which must be made by the airport planner is the amount and type of traffic which is to be served by the terminal. The increased congestion at airports of large cities indicates that it may be advisable to provide separate terminals to serve each of several classes of planes. This requires a classification of airports based upon the services rendered by the planes using the airport and an analysis of the requirements of each type of plane used as well as the study of the amount of present and future traffic to be expected. A careful consideration of present-day planes and performances should be made as well as an attempt to predict future requirements. The airplane industry is relatively new and it expands very rapidly, particularly in wartime.

The loads which a pavement must support are determined from the weight of the plane and the number and size of tires. Therefore, the engineer should know the type of plane which is to use the airport as well as the number of planes per day which will probably use the runways. This latter information is important as the number of repetitions

TABLE II

Airport Size Standards for Use in Planning

	Class 1	Class 2	Class 3	Class 4 and 5
Type of community	Small communities not on present or proposed air carrier system. Includes communities up to a population of approximately 5000.	Larger communities located on present or proposed feeder line airways and having considerable aeronautical activity. General population range 5000 to 25,000.	Important cities on feeder line airway systems and many intermediate points on the main line airways. General population range 25,000 to several hundred thousand.	Cities in this group represent the major industrial centers of the nation and important junction points or terminals on the airways system. Class 5 same as Class 4.
Type of plane which airport may safely accommodate	Small private-owned-type planes up to gross load of 4000 lb. or those whose wing loading × power loading does not exceed 190.	Large private planes and some small transport planes in gross weight range between 4000 and 15,000 lb., or having a wing loading × power loading of 190 to 230.	Present-day transport planes between 10,000 and 50,000 lb. gross weight, or having a wing loading × power loading of 230 and over.	Largest planes in use and those planned for immediate future with a gross weight of 50,000 lb. and over, or a wing loading × power loading of 230 and over. Class 5 same as Class 4.
Length of landing strips *	1800 to 2700 ft.	2700 to 3700 ft.	3700 to 4700 ft.	4700 to 5700 ft. Class 5 5700 ft. and over.
Width of usable landing strips	300 ft.	500 ft.	500 ft.	500 ft.
Length of paved runways	None	2500 to 3500 ft.	3500 to 4500 ft.	4500 to 5500 ft. Class 5 5500 ft. and over.
Width of paved runways	None	150 ft. night operation. 100 ft. day operation only.	200 ft. for instrument landing. 150 ft. night operation. 100 ft. day operation only.	Same as Class 3
Number of landing strips and runways determined by percentage of winds, including calms, covered by landing strip and runway alignment †	70%	75%	80%	90%
Distance between center lines of parallel runways		700 ft. minimum	700 ft. minimum	700 ft. minimum
Distance between center line or runway and airport buildings. Instrument landing runways		750 ft. minimum	750 ft. minimum	750 ft. minimum
Distance between center line of runway and aprons and loading platforms. Instrument landing runways		500 ft. minimum	500 ft. minimum	500 ft. minimum
Distance between center line of runway and airport buildings (other runways)		350 ft. minimum	350 ft. minimum	350 ft. minimum
Distance between center line of runway and aprons and loading platforms and parking areas (other runways)		250 ft. minimum	250 ft. minimum	250 ft. minimum
Landing strip and runway grades—transverse	2% maximum	2% maximum	1½% maximum	1½% maximum
Landing strip and runway grades—uniform longitudinal	2% maximum	1½% maximum	1½% maximum	1% maximum
Grade breaks—longitudinal ‡	Minus 1½% to plus 1½% maximum	Minus 1¼% to plus 1¼% maximum	Minus 1% to plus 1% maximum	Minus 1% to plus 1% maximum
Static design loads for runway and apron paving based on present-day aircraft. Load considered distributed equally between two main wheels or sets of wheels	No paving recommended.	30,000 lb.	74,000 lb.	120,000 lb.
Probable future (10 years) maximum static gross loads to be considered in the design or runway and apron paving of drainage structures	20,000 lb.	60,000 lb.	150,000 lb.	300,000 lb.
Probable range of static airplane tire pressures	10 to 25 lb./sq. in.	15 to 50 lb./sq. in.	30 to 75 lb./sq. in.	50 to 85 lb./sq. in.
Facilities required	Drainage, fencing, marking, wind direction indicator.	Marking, wind direction indicator, drainage, lighting, hangar and shop, fueling, fencing, weather information, office space.	Marking, wind direction indicator, drainage, lighting, hangar and shop, fueling, fencing, Weather Bureau, two-way radio, visual traffic control, instrument approach system—when required, administration building.	Same as Class 3
Width of taxiways		50 ft. minimum	50 ft. minimum	50 ft. minimum
Distance between runway center line and parallel taxiway center line		275 ft. minimum	275 ft. minimum	275 ft. minimum
Distance from boundary fence, obstructions, and so forth, to taxiway center line		100 ft. minimum	150 ft. minimum	150 ft. minimum. Class 5 200 ft. minimum.
Longitudinal grade		3% maximum	2½% maximum	2½% maximum
Transverse grade		1½% maximum	1½% maximum	1½% maximum
Angle of taxiway intersection with runway ends		60° minimum	60° minimum	60° minimum

* All the above landing strip and runway lengths are based on sea level conditions; for higher altitudes increases are necessary. One surfaced runway is recommended for the full length of each landing strip for airports in Classes 2, 3, 4, and 5.

† Landing strips and runways should be sufficient in number to permit take-offs and landings to be made within 22½ degrees of the true wind direction for the percentage shown above of winds over 5 m.p.h., based on at least a 10-year Weather Bureau wind record where possible.

‡ Longitudinal intersecting grades on a runway or landing strip should be joined by a vertical curve at least 500 ft. in length. It is also recommended that the tangent interval between the point of tangency of one curve and the point of curvature of the succeeding curve be not less than 1000 ft. If economically practical, grade breaks should be so controlled that the sight line will be unobstructed from any point 10 ft. above the surface of the runway to any other point 10 ft. above the runway. In general, there should be no change in landing area grades of more than ½% in any 100-ft. interval.

This table is based on a group of tables compiled by the Airport Section, Technical Development Division, Civil Aeronautics Administration, Washington, D. C., and published in "Airport Design Information."

Runway grades should not be altered to accommodate taxiway intersections or connections. At large airports where traffic is heavy it may be advisable to construct a warming-up apron and by-pass on taxiways connecting to the ends of runways.

Taxiways should not connect to the ends of runways at an angle of less than 90° to incoming traffic.

of traffic on a pavement enters into the design, as will be explained later.

The airport classifications given in Table II are closely connected with the method used in rating aircraft. The lifting power of a plane will vary with its wing loading and power loading. The wing loading is determined by dividing the gross weight of the plane by the total wing area. The power loading is determined by dividing the gross weight by the total available horsepower of its engine or engines. A combination of these two factors gives an index number commonly used for airplane classification. This index number is the product of the wing loading and the power loading.

In general, the smaller planes having index numbers up to 190 can be accommodated on what are shown as class 1 airports in Table II. Class 2 airports will accommodate planes with index numbers between 190 and 230. Planes with index numbers over 230 will require airports of either Class 3, 4, or 5. Table III gives a list of some planes to illustrate the method of listing the index numbers.

The amount of traffic in the different classes should be determined to the best of the designer's ability. From this determination the classes of traffic representing the major number of landings and take-offs will form the basis upon which the airport facilities will be provided.

For military airports the coordinated plan of the armed forces will have fixed the type of airport to be constructed in various sections of the country. This will be controlled by the strategic location of the area in the protection of the country. The requirements for capacity and size are decided upon by those familiar with the strategy to be carried on by our fighters in the air. The civilian designer will not be called upon to fix the needs of this type of airport but may be required to study possible locations where airports of a given type may be located to satisfy the specifications set up by military authorities.

Information to be used as a guide in determining the probable amount of civilian traffic for a given area will be less specific than for an airport designed to serve some definite military purpose. By investigating records of other airports in the vicinity or in similar localities, and consulting existing air line records the growth and present traffic may be estimated. A study of postal reports, retail business reports and census figures will aid in forecasting the probable future growth of passenger, mail, and freight traffic.

In giving consideration to the probable amount of freight traffic at an airport it must be remembered that the major contribution of air transportation is the item of speed. The costs of moving large tonnages of freight by plane are many times that of like transportation by rail or sea. Although larger cargo airplanes will be constructed and should be considered when planning the airport it would be safe to assume a conservative point of view by visualizing

TABLE III

AIRPLANES ACCOMMODATED BY CLASS 1 AIRPORTS

(Below 190)

Company and Model Designation	Wing Loading (lb./sq. ft.) × Power Loading (lb./hp.)	Gross Weight	Horsepower
Abrams Explorer T-2	138	4,000	450
Curtiss-Wright A-19-R	110	2,837	420
Curtiss-Wright CW-22 Sport	140	3,200	420
Piper Cub Coupe	157	1,200	50
Piper Cub Sport	136	1,100	50
Piper Cub Trainer	140	1,000	40
Ryan S-T-4 Special	138	1,600	150
Ryan S-C	158	2,150	145
Waco ZKS	152	3,250	285

AIRPLANES ACCOMMODATED BY CLASS 2 AIRPORTS

(190 to 230)

Company and Model Designation	Wing Loading (lb./sq. ft.) × Power Loading (lb./hp.)	Gross Weight	Horsepower
Boeing 247 D	202	1,690	90
Fairchild 24	227	2,550	165
Fairchild 45	201	4,000	320
Waco EGC	206	3,800	320
Waco ZVN	198	3,650	285

AIRPLANES ACCOMMODATED BY CLASS 3, 4, AND 5 AIRPORTS

(Over 230)

Company and Model Designation	Wing Loading (lb./sq. ft.) × Power Loading (lb./hp.)	Gross Weight	Horsepower
Douglas DC-2	246	18,560	1500
Lockheed Electra 10E	268	10,500	900
Lockheed 12A	236	8,650	800
Boeing 307	378	45,000	2200
Curtiss-Wright CE-20	299	63,000	3200
Douglas DC-3 SIC 3-G	268	25,200	2100
Douglas DC-4	356	52,000	4400
Lockheed 14 G-3 B Provisional Gross	328	17,500	1700
Lockheed 14 H Normal Gross	280	15,500	1700

the cargos as consisting of the lighter, more perishable class of freight where speed of delivery makes it economical to ship by air. The future freight transportation by air does not assume the proportions of a threat of placing our present land and sea facilities in obsolescence but rather points toward a favorable augmentation of these facilities.

Experience at the larger airports of the country indicates that the volume of traffic which may be handled is limited by the number of parallel runways and the capacity of the control tower safely to guide the planes in and out of the terminal. This capacity has been found to range from 24 to 30 operations per hour depending upon the arrangement of runways and taxiways available to provide clear landing strips for each operation. All-over landing areas which permit landing and take-off in any direction are more efficient for mass take-offs as required in military operations where formation flying is practiced.

14. Flight Requirements Affecting the Size of the Airport. A study of Table II will show the length and width of landing strips recommended for each type of traffic which may be using the airport. This will serve as a guide in deciding upon the amount of land which must be secured to provide for the class of traffic to be served.

The lengths shown in this table are for conditions existing at sea level. It must be remembered that the lifting capacity of a plane depends upon the density of the air. This density is affected by altitude and temperature. In regions where the barometer readings are consistently low, because of weather conditions or high altitude, a longer take-off distance will be required. An altitude of 6000 feet will cause a decrease in the rate of climb to such an extent that the runways required will be about 1.5 times longer than at sea level for the same class of airplanes. An area in which high temperatures are prevalent will also require longer runways as a result of the lesser density of the air. The maximum temperatures and the minimum barometer conditions will ordinarily require an increase in the values given in Table IV.

TABLE IV

EFFECT OF ALTITUDE ABOVE SEA LEVEL ON AIRPORT SIZE

This table gives the recommended runway and landing strip lengths in feet for elevations above sea level for the four airport-planning classifications.

Elevation	Class 1	Class 2	Class 3	Class 4	Class 5
Sea level	1800	2700	3700	4700	5700
1,000	2050	2950	3950	4950	5950
2,000	2300	3200	4300	5200	6200
3,000	2550	3450	4450	5450	6450
4,000	2800	3700	4700	5700	6700
5,000	3050	3950	4950	5950	6950
6,000	3300	4300	5200	6200	7200
7,000	3550	4450	5450	6450	7450
8,000	3800	4700	5700	6700	7700
9,000	4050	4950	5950	6950	7950
10,000	4300	5200	6200	7200	8200

Note. Runway or landing strip lengths for intermediate elevations may be obtained by interpolation. The values in the above table were computed from the formula $L = L_S + \frac{A}{4}$, where L = the length required at the site; L_S = the recommended length at sea level; A = the altitude of site above sea level.

15. Wind. The direction and velocity of the wind are important factors in the selection of a site for an air terminal. The effect of the wind velocity upon the take-off distance is not actually considered in selecting runway lengths but it does offer a safety factor which is worthy of note. Each plane must obtain some specific ground speed before take-off. If runways are provided long enough for this speed to be attained in a calm then any wind velocity opposing the direction of take-off will shorten the required run. For example, a plane taking off into a 5-mile-per-hour wind will lift off the ground at a speed 5 miles per hour less than that required in a dead calm. The distance will therefore be shorter.

16. Type of Surface. Another factor having a minor effect upon the required take-off distance is the relative friction offered by different runway surfaces. Natural earth, turf, and gravel will offer greater resistance to the speed of a plane than hard-surfaced runways of asphalt or portland cement concrete.

17. Buildings. The length requirement will in general decide the size of the area needed for the airport. The space needed for buildings, storage, etc., will in most cases be found along the margin of the landing area. Exceptions to this will be such military bases as large training centers where additional space will be needed for barracks, school buildings, drill area, etc.

18. Obstacles. The discussion of size of area needed would not be complete without mention of the limiting effect of high obstacles in the area surrounding an airport. Each class of plane has a maximum rate at which it can climb after take-off. Any high object in the path of take-off will therefore limit the location of the end of the runway used in that direction. The effective landing area on any given site will therefore be limited by such obstructions. A more detailed discussion of obstructions will be found in Art. 26.

19. The Site. After studying the nature and amount of traffic to be provided for, the airport designer should next study all available locations large enough to accommodate the facilities needed. There are several general questions which this study should answer:

1. Which areas are consistent with the city and regional plan?
2. Which areas lend themselves most readily to proper grading and drainage with a reasonable expenditure of money?
3. Which areas satisfy the flight requirements of the planes using this terminal?

Many available land areas will be eliminated because of their failure to satisfy one or more of the above conditions. Of those areas remaining the final choice must be made by giving careful consideration to the factors affecting each of these conditions.

20. Fitting the Airport to the Regional Plan. The site for an airport should be so located that adequate area will be available for future expansion. Its location should not be contrary to established zoning ordinances under which existing development of the area has taken place. The area surrounding the site should be such that probable airport zoning restrictions will not affect its usual development. Gas, electric, telephone, water, and sewage facilities should be available whenever possible but their absence should not condemn an otherwise satisfactory location. These facilities can ordinarily be extended or provided without prohibitive costs.

The relationship of the airport to surface transportation facilities is of major importance. The air terminal is a point of transfer of passengers and goods to other means of transportation.

The location of an air terminal with respect to the hotel and business centers of the community should be measured in time rather than in distance. A desirable location would be such that no more than 15 to 20 minutes would be required to reach the airport from the business center of the community. This can be accomplished by location near existing rapid transit lines and high speed highways by which passengers and goods may be moved rapidly to their destination in the area.

The available area and its relationship to major highways must be studied in order that facilities for busses, taxis, private cars, and trucks may be provided in such a way that steady flow of traffic may be maintained without congestion in parking or transfer of passengers and goods.

A busy airport attracts many visitors who enjoy watching the arrival and departure of planes. In this respect an airport may be considered an addition to the recreational facilities of a community. Consideration should be given this phase of the development by utilizing adjacent areas as part of the recreational system of the city or region. Such development adjacent to the airport helps prevent the erection of obstacles outside the landing area and may provide added space for expansion should it become necessary. Further thought should be given to the airport project as a community center by utilizing some portion of the terminal building as an attractive restaurant which will undoubtedly be patronized by other than the plane passengers. Such a restaurant should be placed where a good view of the landing area may be had by those dining within. The adaptability of each site to such a plan of augmenting the recreational facilities of the community should be considered before a location is decided upon.

21. Topography as It Affects the Site Selection. The topography of each site and that of the area surrounding each site within about a three-mile radius should receive careful study. Topographic maps of the vicinity to a scale of 1000 feet to the inch are most useful for this study. Aerial photographs used in stereographic pairs are an excellent means of determining the nature of the topography. These give a better means of studying the land coverage such as brush, heavy woods, and type of cultivation; they also show possible obstructions in the vicinity.

The study of topography should include a comparison of each site as to ease of drainage, probable grading required, and obstructions in the vicinity which would limit the effective length available for take-off and landing in each direction.

A map should be prepared for each area upon which the elevations of all hills, buildings, electric power lines, and any other obstructions within a three-mile radius are shown. That area having the smallest number of obstructions in the probable direction of the runways will be best suited from this point of view. A detailed method of study of the effect of these obstructions is given in Chapter III.

Other items of interest to the airport planner are the wind conditions, prevalence of fog, and the smoke conditions from industrial centers, which exist at each site.

In general the wind directions and intensities will be quite uniform in a given area unless changed by predominating hills and ridges. Purely local air currents should be determined for each site.

The full-time operation of an airport may be materially impaired by the prevalence of fog, which obscures the landing area. In this respect airports located on high ground are less apt to be hidden by fog. Sites which are located on low ground and those which are to leeward, with respect to prevailing winds, of swampy areas are apt to be undesirable from this point of view.

A similar condition is created by smoke from industrial areas and to avoid such obscurement of the landing area sites which are to windward of such industrial plants should be selected.

The final selection of the site to be used should be based on the thorough analysis of all the above considerations and the best judgment of the airport planner. Too much emphasis cannot be placed upon the value of extensive, thorough investigations preliminary to final decision. The first cost of land procurement and construction as a determining factor should always be tempered by the future possibilities of the site.

Chapter III

Planning the Site Selected

22. Factors in Site Planning. After the available sites in the area have been carefully analyzed and one site which will serve as an air terminal for the community has been decided upon, the work of planning a site must be considered. As in selecting the site, the future development of the site must be continually kept in mind while the layout of the facilities is being planned. The ultimate plan should be the factor controlling the layout even though present traffic and funds may dictate a much less pretentious development. The airport thus planned will survive the rapid growth and changes in air transportation whereas others may soon become obsolete because of the failure of the designer to allow his imagination to carry him into the future and provide for expansion which may seem merely a dream of the future.

Although the landing area is of major importance, there are many other services which must be provided at a completely efficient air terminal. The wise airport engineer will welcome the opinions of individuals accustomed to working at air terminals. Service men, pilots, traffic managers, postal and express authorities, commercial air line officials, and military authorities will all be able to contribute suggestions which will be of value in creating a well-coordinated plan.

The planning of the airport site must include:

1. The proper orientation of runways if an all-over landing area is not being used.
2. The pattern of the runway and taxiway strips to assure easy circulation of traffic.
3. The location of buildings relative to the landing area to assure coordinated movement of planes, passengers, and freight.
4. The location of highway approaches and parking areas for cars, busses, and trucks.
5. The location of lighting facilities.
6. The architectural features of buildings and landscaping to provide a terminal which will be attractive to passengers and other visitors to the airport.

This embraces a wide field and in general will require the combined efforts of several individuals, each proficient in his own branch of engineering or architectural planning.

As a preliminary step in the planning operation, the engineer should furnish himself with maps upon which the plan may be superimposed. A map of the vicinity showing the area within at least 3 miles of the airport site to a scale of 800 or 1000 feet to the inch, with 20-foot contours shown, and also a map of the site to a scale of at least 200 feet to the inch, with 2-foot contours shown, will be found valuable tools for the airport planner. Aerial photographs will also convey added information regarding the character of the ground cover over the site and adjacent areas.

Records of the direction, force, and duration of the wind in the vicinity and the fog characteristics of the area should be secured from weather observatories near the location of the airport. The longer these records have been kept the more reliable will be the data secured.

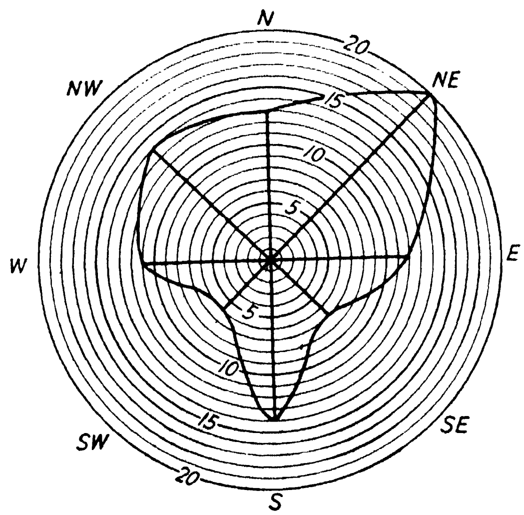
23. Location of Runways. The direction, number, and pattern of the runways will be affected by many factors. Each of these will be closely allied with the others and must be considered simultaneously when locating the runway strips upon the map of the site. For simplicity of explanation each will be discussed separately on the pages which follow.

These factors are: the wind, the topography of the site, obstructions adjacent to the airport, approach zones, the free circulation of traffic, and the future expansion of the airport.

24. The Wind and Its Effect upon Runway Orientation. The available wind data should first be used to construct

what is known as the wind rose. This is a diagram which shows the wind direction and the percentage of time the wind may be expected to blow from these directions. Winds of less than 4 miles per hour are considered to be dead calms. The percentages shown about a wind rose will therefore usually add up to less than 100 per cent of the time, the difference being the per cent of time during a year when dead calms may be expected.

Figure 6 illustrates the construction of a wind rose from the wind data. The wind rose should then be oriented upon the topographic maps to be used in planning.



Wind Data

(The average of ten years' observation.)

Direction from	% of Time
North	13
N East	20
East	12
S East	7
South	14
S West	6
West	11
N West	14
Less than 4 m.p.h.	3

FIG. 6. Wind rose diagram.

The directions of the runways to be used will next be determined. These should allow landings in either six or eight directions requiring either three or four strips intersecting in some general pattern.

These strips should be so oriented that the take-off and landing of planes may be accomplished by heading into the wind under most of the variable wind conditions. Specifications require that these runways be so oriented that one may be selected for use with a maximum variation from a head wind of 22½ degrees for at least 75 per cent of the time. More rigid specifications require that this condition be met for as much as 90 per cent of the time. By using the wind rose, the direction of either three or four lines which will satisfy this requirement may be determined.

The runways must be laid out on the map parallel to these lines. By so orienting the landing strips there will be only a small chance that pilots will ever have to land or take off into a wind whose direction varies more than 22½ degrees from the axis of the runway.

25. The Topography of the Site. The cost of construction is greatly dependent upon the topography. This affects the amount of grading which must be done and the drainage system which must be employed to assure a stable soil under all weather conditions.

When the landing strip arrangement is being planned, the relative amount of grading involved for different positions of the runways may be estimated from the contours shown on the map. The location of the landing strips should be selected so that the natural drainage of the area will be disturbed as little as possible. In rolling or hilly country this will be more difficult than where the site is located in flat country.

26. Obstructions Adjacent to the Site. As planes approach or leave an airport the gradual climb or descent requires that the area surrounding an airport be free from obstructions which would project into the glide path of the planes using the airport. Glide path ratios have been set for safe operations of various classes of traffic. These ratios are 20 to 1 for Class 1 airports, 30 to 1 for Class 2, 3, 4, and 5 airports, and 40 to 1 for runways where landings will be made while operation is entirely by instruments. These represent the horizontal distance covered for each unit of vertical rise.

These ratios are used to examine the effect of obstacles adjacent to the landing area upon the runway locations. To illustrate this let us assume that a stack on a factory building is 90 feet above the elevation of the landing fields and in the vicinity of the field. For instrument landings using a glide ratio of 40 to 1 the nearest end of a runway should be at least 3600 feet from the stack. Similarly, for Class 1 airports using a glide ratio of 20 to 1 the take-off may be made at a distance of 1800 feet from the stack.

When all-over landing fields are used the entire area surrounding the site should be examined to make certain that nothing projects into the glide paths.

27. Approach Zones. When landing strips are used, wide clearance areas are provided as approach zones. These zones are trapezoidal areas having a width of 500 feet at the boundary of the airport or landing field and widening to 2500 feet at a distance of 2 miles from the boundary of the airport. The center line of these trapezoids is the extension of the center line of the runway.

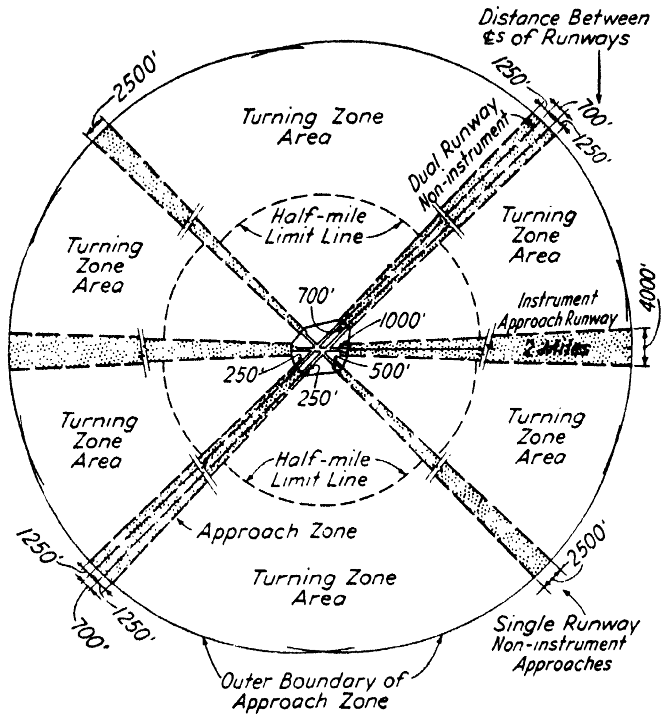


FIG. 7. Approach zones.

For instrument runways the width of these approach zones are widened to 1000 feet at the boundary and 4000 feet at the 2-mile point.

28. Free Circulation of Traffic. In order to avoid congestion of traffic the pattern of runways and taxiways should be such that the maximum capacity of the traffic control facilities may be effective without the necessity of holding incoming planes in the air while others are taking off. The system should permit each plane, as it lands, to taxi to the loading area without returning along the landing strip. This immediately frees the runway for other planes. This circulation may be accomplished by using the other runways or providing taxiways by which the plane may taxi to the terminal from the end of any landing strip.

29. Future Expansion. It is quite difficult to predict just what the plane of the future will weigh or what will be its operating characteristics. It is certain that planes will be much larger than those of present design and it is probable that with an increase of size will come a corresponding increase in the size of the airports to accommodate such planes. Therefore, an engineer should have this in mind when selecting the site and planning the layout. Where possible, a larger area should be purchased so that extensions of runways is possible. It is not uncommon to find a demand for runways of 8000 feet and 10,000 feet, and even some for 11,000 feet. This presents a difficult problem to the designing engineer for, if he must construct two or three runways of nearly 2 miles each, he may have trouble in finding an area large enough to combine these landing strips all in one area. Some engineers are making plans to use separate areas for each runway when it is difficult to find an area large enough to accommodate all directions of runways chosen. Although this would increase the problem at the terminal in handling the traffic it may be the solution for some cities and communities. These areas may be separated by several miles with only a single runway on each area.

30. A Suggestion to the Planner. A practical approach to the proper orientation of landing strips is to prepare to scale strips of tracing paper of the required length and width for the type of traffic to be served. These transparent strips representing the landing strips may then be placed over the topographic map and moved about until the best location is secured. By noting the elevations from the contours and maintaining the directions selected on the wind rose diagram, very satisfactory locations will be selected.

In a similar manner templates of the necessary buildings may be moved about the map of the area in order that a visual study may be made of the location of each building so that its functions may be most efficiently realized.

31. Runway Numbering System. The present system of numbering runways is illustrated in Fig. 8. The numbers are painted on the runways and are about 50 feet in height. These numbers are the magnetic directions with the last figure (usually 0) dropped. For example, a direction of 330 degrees would give a runway number of 33 and a direction of 150 degrees would give a runway number of 15. The number corresponds to the magnetic heading of the approaching plane.

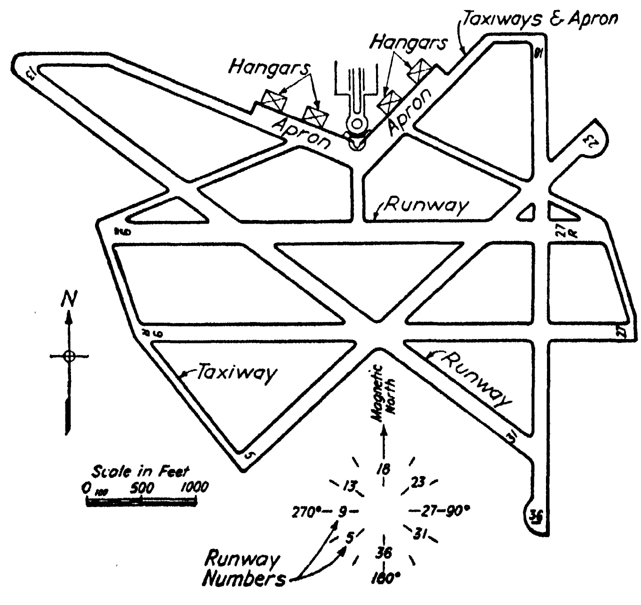


FIG. 8. Runway numbers.

32. Utilities. Airport buildings must have electric power, gas, water, sewers, and communications. These facts should be kept in mind when a site is selected and an attempt should be made to use existing facilities as much as possible, although they are secondary considerations.

Large amounts of electric power are required at an airport for lighting and the operation of shops and other services. A plant of considerable capacity is required if a private installation is necessary.

A safe water supply is essential and an extension of the service of the community is usually necessary. A private supply may be installed but this is expensive and sometimes impossible.

The sanitary sewer system may be connected to an existing installation or private septic tanks may be installed. If private sewers are used, the effluent should be carried away from the runways as the moisture might affect the stability of the subsoils. Large amounts of storm water must be carried off and a separate system should be carefully designed to accommodate this water. The design of these systems is taken up in Chapter VI.

It is not usually difficult to tie the airport in with existing communications such as telephone and telegraph but these items should be kept in mind.

33. Building Locations. Many buildings are required at an airport and much thought should be given to their location. The operating efficiency of the airport depends largely upon the general plan. This is particularly true of commercial airports. In military airports, the efficiency of operation may be sacrificed to obtain better concealment and a wide dispersion of the buildings. The buildings at a commercial airport serve several purposes. The following list will give a general picture of the requirement.

1. The administration building, in which provision is made for
   a. Passengers (waiting rooms, ticket office, rest rooms, cafeteria or restaurant, check rooms, concession stands).
   b. Offices for the administrative staff.
   c. Post office for handling air mail.
   d. Weather bureau office.
   e. Radio rooms for communications.
   f. A traffic control room.
   g. A traffic control tower.
2. Hangars.
3. Repair shops.
4. Parking areas for employees, passengers, freight and spectators.

Buildings should be located centrally with respect to the runways. Such a location will reduce the amount of maneuvering of the planes on the field in order to discharge and take on passengers and freight.

The size of the administration building will vary with the amount of traffic. Some airports have large well-equipped rooms for the comfort and convenience of the passengers. Waiting rooms and restaurants usually face the air field and at large airports the restaurants have been made places for dining and entertainment. The type of traffic is similar to that of the railroads and a study of their problems will give the designer some basic condition on which to plan.

34. Parking Areas. Parking areas are not classed as buildings but the locations of buildings often control the space available for parking. Many airports of the future will be centers for entertainment and ample parking space will add to the attraction of the place. The parking area should be located so that spectators may see the field from parked automobiles. There will be an increasing amount of certain types of freight carried by air and provision must be made for trucks and busses.

Separate parking areas near the shops should be provided for employees.

35. Airport Lighting Facilities. The Civil Aeronautics Administration specifies the requirements for an adequately lighted field. These include the spacing, location, and number of all the necessary lights. The planner must prepare a map showing the location of the lights in accordance with these specifications. It will ordinarily be necessary to provide a building or space in another building for the necessary controls and other electrical equipment. Lighting is discussed in detail in Chapter X.

36. Camouflage. A military airport in combat zones must be made as inconspicuous as possible. This is particularly true of the areas used for storing planes. The runways are so long that it is quite difficult to make them invisible from the air by any form of cover, but color may be selected which will help to camouflage the airport.

There are a few fundamentals which may be kept in mind when planning a military airport. They are summarized in the following statements.

1. Try to preserve as many trees as possible.
2. Avoid regularity in design.
3. Distribute structures and storage space for planes (dispersion).
4. Attempt deception by coloring.
5. Cover with artificial covering.
6. Utilize the topography for underground installations.

The methods used in camouflage will depend upon the type of attack which is expected. If an airport is in a combat area it may be subjected to strafing and fire from low-flying planes at altitudes of 1000 feet or less. If the airport is back from the front line of activity it will probably be subjected to high altitude precision bombing. These altitudes are usually above 10,000 feet. Machine guns, cannon, and bombs are directed toward their target

by the use of sights and the human eye must do the sighting. Maps and aerial photographs will assist the plane crew in locating the general position of an objective but, in the final analysis, someone must see the objective before he can direct the fire. This means that a person in a low-flying plane will see more detail than a person flying at high altitudes.

It is true that a person in a low flying plane will have less time to make decisions but he is closer to the target and therefore greater care must be taken in concealing strategic areas. He may detect imperfections in the camouflage more readily than he could from high altitudes.

A few seconds' delay in the detection of an object by the bombardier may be the difference between a hit and a miss. He has no more than a minute to direct the plane, sight, and set all mechanisms. Approximately the first 25 seconds will be used in directing the plane on the correct course and the next 35 seconds in setting his sights on the target. Therefore, any delay which may cause doubt in the mind of the bombardier as to the location of the target will materially reduce the accuracy of his aim.

The glare of a runway may be reduced by a bituminous spray and thus blend the runway with the surroundings. Other coloring may be used to accomplish the same results.

Attempts have been made to make the runway look like cultivated fields. This is accomplished by a mixture of colors daubed on in a more or less irregular pattern which blends with the surrounding fields. Shadows may be "telltale" markings which disclose the presence of structures and it has been found advisable to color the ground upon which such shadows fall. This may be accomplished by spraying and dark colors are preferable.

Complete concealment may be attained by stretching a chicken wire net over a structure with a "garnish" to blend with the surroundings. Cotton fabric, burlap, canvas, steel wool, glass wool, rock wool, and chicken feathers have been used for a "garnish" and are all susceptible to coloring.

Care should be taken in the selection of colors used to match the colors of nature. Infrared photography often will disclose a camouflaged object. This is particularly true when green foliage is used. To the eye, this foliage appears green as the green rays predominate but there are red rays, invisible to the eye, which stand out in the infrared photograph. The artificial green does not reflect the red rays. There is therefore a marked difference in the reproduction of the two greens on a photograph.

The visibility of a runway from the air may be reduced by using a granular surface. Pebbles have been used in the surface to produce a dispersion of the light rays. A mottled surface with sharply contrasting colors may be used but runways present a very large surface to treat in this manner. Sometimes contours have been painted or sprayed on the landing area to make the field look like a hill.

Trees and shrubs may be planted and the engineer should save all the existing trees possible.

One of the most difficult objects to camouflage is a body of water. Such an object makes an excellent landmark and, therefore, airports should not be placed near lakes, streams, or other bodies of water.

37. Typical Airport Runway Patterns. Figures 9 through 19 illustrate some of the typical runway patterns which have been used and the relation of the airport buildings to the landing area.

Figures 9 and 10 are illustrations of Bowman Field, Louisville, Kentucky. The aprons in front of the hangars and terminal form taxiway connections to one end of each runway. The runways themselves must be used as taxiways for part of their length when take-offs are made in a north, northwest, and northeast direction.

In Fig. 10 it will be noted that provision has been made for hangar construction parallel to the highway on the north. The location of hangars opposite the end of the north and northwest landing strips might introduce a hazard.

Figure 11 illustrates a partial development in which only the major runway has been completely paved. The ends of other runways were paved at the time of original construction to make later development easier and they also serve as runway markers.

The pattern shown in Fig. 12 leaves ample room for extension of the runways to the size of the one long runway already established.

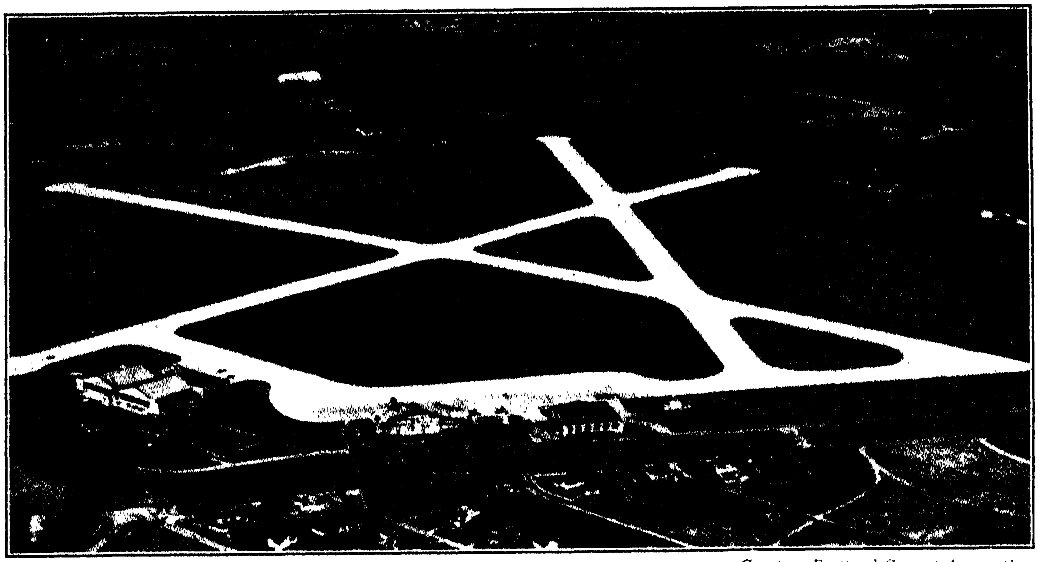
Ample space has been left in the airport shown in Fig. 13 for expansion of hangar facilities along the wide parking apron. This pattern would be improved by taxiways to reach the extremities of the long runways.

Figures 14 and 15 illustrate the use of taxi strips to take care of circulation of traffic without the need of traveling along the landing strips.

The sketch in Fig. 16 shows a grouping of buildings conveniently located with respect to each other and not interfering with runway use. Added connections from the apron to the runway at each end and taxi strips to the northern end of the runways would improve the traffic circulation on this pattern.

Figures 17, 18, and 19 illustrate the step-by-step development which may be made if the ultimate layout is considered at the time of the original purchase of land and the first construction.

Additional suggestions for airport layouts may be found in a bulletin titled "Airport Design," a publication of the U. S. Department of Commerce.



Courtesy Portland Cement Association

FIG. 9. Bowman Field.

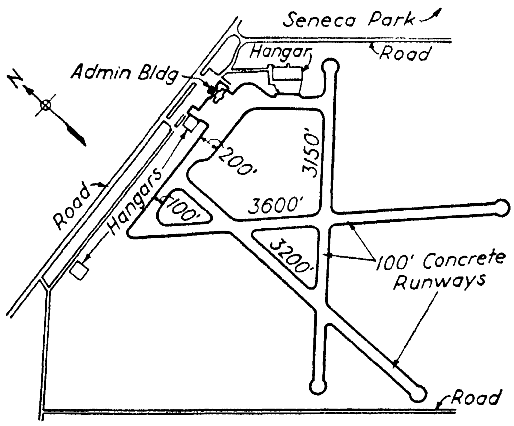
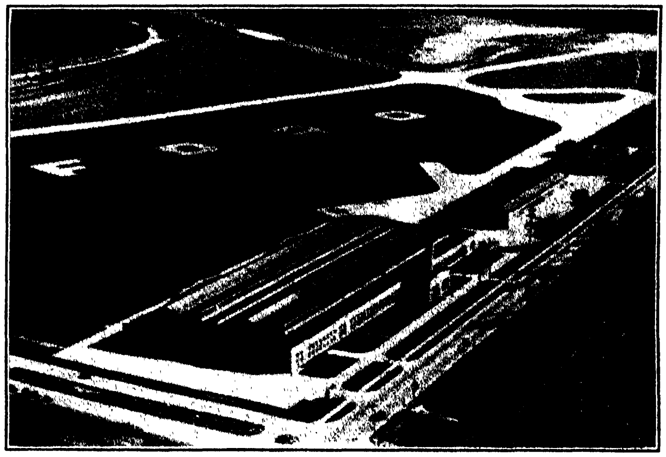
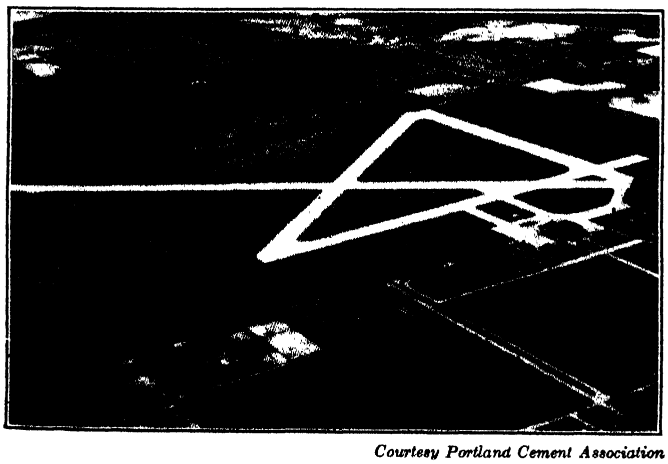


FIG. 10.



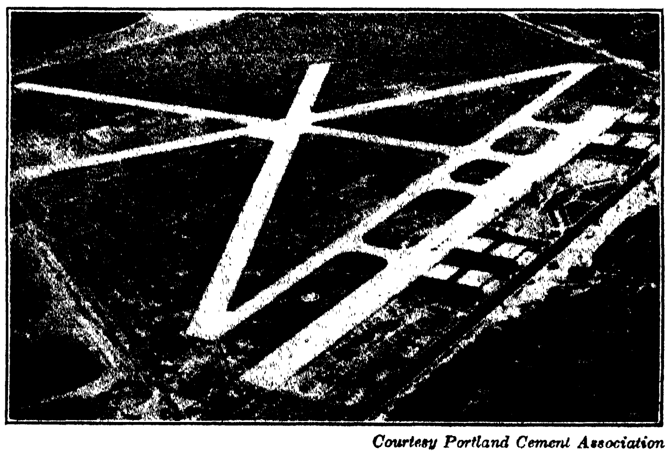
Courtesy Portland Cement Association

FIG. 11. Partially paved runways.



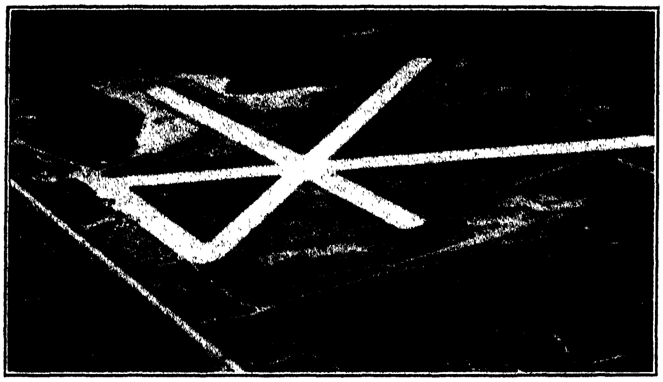
Courtesy Portland Cement Association

FIG. 12. Area partially developed.



Courtesy Portland Cement Association

FIG. 13. Possible future hangar extension.



Courtesy Portland Cement Association

FIG. 14. Use of taxi strips.

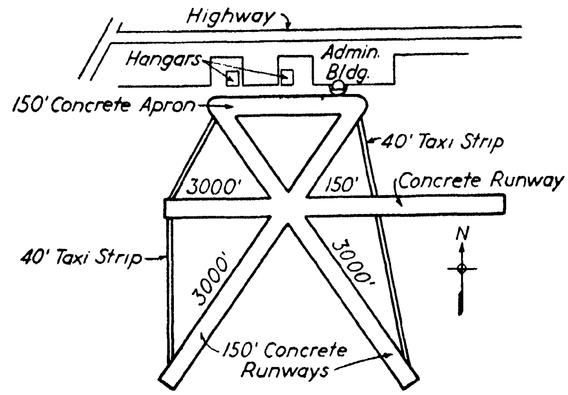


FIG. 15. A runway pattern with taxiways.

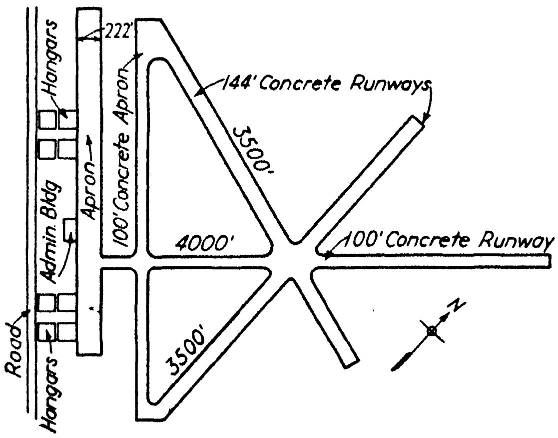


FIG. 16. Typical building location.

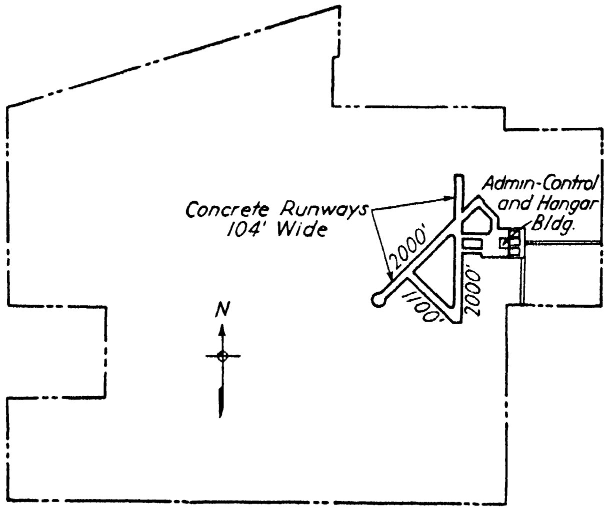


FIG. 17. Initial step in expanding an airport.

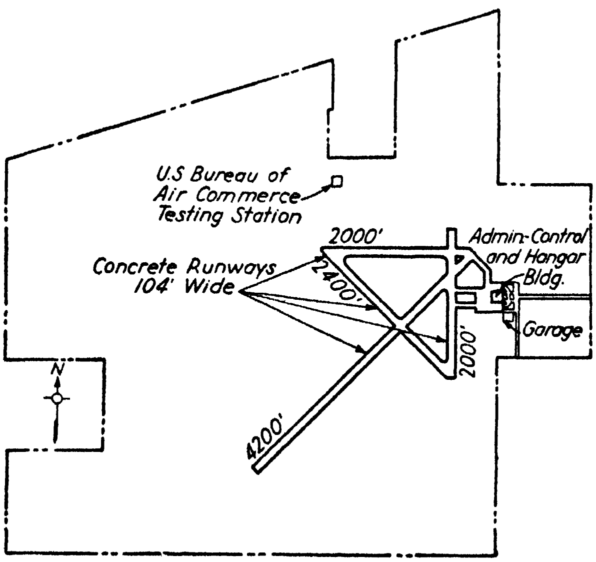


FIG. 18. Second step in expansion.

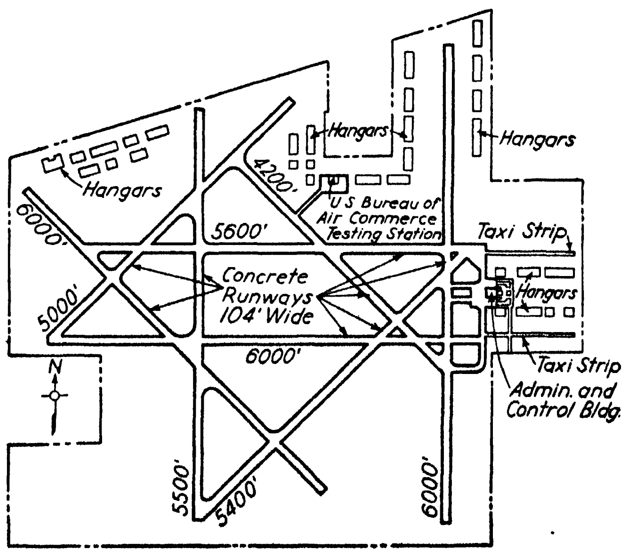


FIG. 19. Ultimate development of area.

Chapter IV

Surveys Preliminary to Design

38. Maps, the Basis for Design. The designing engineer must depend upon the field data of the survey party for the fundamental information upon which the final design will be based. Surveys should be directed by an engineer familiar with the various phases of airport planning and construction so that the surveys will provide adequate maps for the designing engineer.

39. Selecting the Site. The designing engineer wishes to study the location of possible sites in relation to the entire community and to do this he should have a small scale map of the area which shows the general topography, the location of highways, railroads, and waterways in the vicinity; the location of other cities and other air terminals; the relation of the site to existing airways; and other features already discussed in relation to the site selection.

40. Topographic Maps. There are probably topographic maps of the Geological Survey available if the airport is to be constructed in the United States. These will serve for preliminary planning and may be supplemented by aerial photographs of the area with a few field surveys and inspection trips on the ground. The regional and sectional maps of the Civil Aeronautics Administration are also valuable for the preliminary study.

41. Selecting the Runway Pattern. The designing engineer will require several facts which will govern the general pattern and direction of runways. The best available source should be consulted to obtain information on wind direction and intensity. The weather stations operating in nearby cities and at other airports in the locality are sources for this information.

The obstructions which may influence the location of approach zones must be determined. These will be natural high points in the vicinity, buildings, transmission lines, etc.

The small scale maps previously used will be sufficient for the selection of the general plan of the runways. The preliminary study should include the area surrounding the airport for a distance of at least 3 miles in all directions to provide for an analysis of the approach zones to determine topographic obstructions or structures which may be classed as obstructions. It will be necessary to make field measurements of the height of buildings and of the location and elevation of new structures within this circumference which do not appear on the small scale maps. The data secured will be used to decide upon the proper orientation of the runways to secure the approach zone with the least interference.

A topographic map will be required for planning the final location. This requires a map of larger scale showing contours at small intervals and all cultural detail within the immediate area.

A topographic map to a scale of 1 inch = 200 feet with 2-foot contours is best for the final plan. Such a map will allow scaling of distances and elevations which can be used in computations and planning. It will give a base upon which the engineer may superimpose all runways, structures, wind rose data, lighting systems, drainage, and the final contours. Such a map is the work sheet for the whole project.

Plane table surveys will give the best results where a map is to be made for this purpose; however, a transit stadia survey may be used or the area may be blocked out in a square grid system and elevations of the corners may be determined by leveling.

If a grid system is used, it will be best to make one axis of the grid squares parallel to the main runway of the airport. The other axis will be at right angles to this; the elevations parallel to the main runway may be used for profile studies and the elevations at right angles to this may be used for cross-sectional elevations or cross profiles.

42. Profiles. Certain limitations on grades for runways are imposed and profiles of the center lines of each runway are necessary. The planning of a grade line is similar to the method used on highway and railroad work. The scale for profiles should be such that estimates may be made from measurements taken from the drawing. As a rule, the profile of a runway site will be flatter than those used on highway and railroad work. Usually the flattest section of the area is selected for an airport to reduce the amount of excavation, although it may complicate the drainage problems

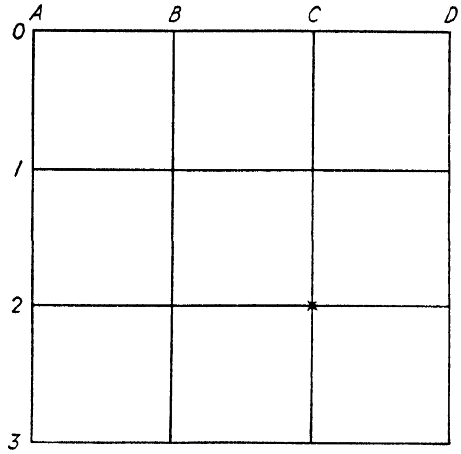
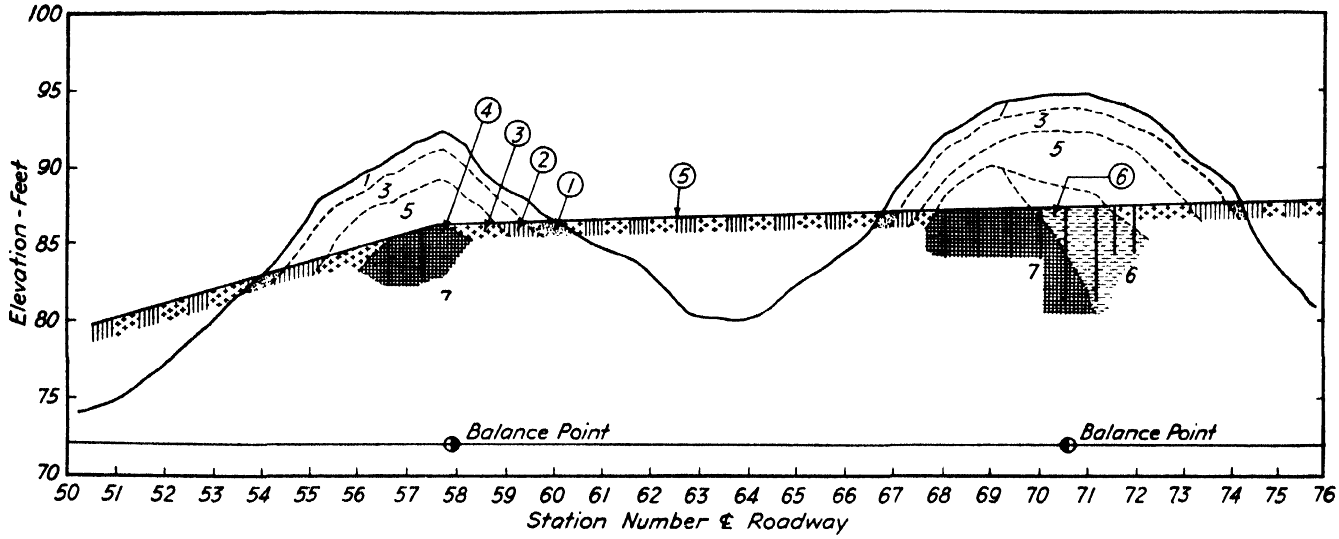


Fig. 20. Grid system.



Legend and Laboratory Test Results

Legend	Mechanical Analysis: Percent of Particles having Diameters Smaller Than 2.0mm.	0.5mm.	0.25mm.	0.05mm.	0.005mm.	0.001mm.	Physical Test Constants: Liquid Limit	Plasticity Index	Shrinkage: Limit	Shrinkage: Ratio	Moisture Equivalent: Centri.	Moisture Equivalent: Field	U.S.B.P.R. Soil Group and Textural Classification
A Horizon, Layer 1, (1)	100	99	98	75	15	4	38	11	26	1.5	35	30	A-4, Silt Loam
B Horizon, Layer 3, (2)	100	99	90	60	27	15	45	25	18	1.8	35	30	A-7, Clay Loam
C_1 Horizon, Layer 5, (3)	95	90	70	46	18	7	25	7	20	1.7	20	23	A-2, Sandy Loam
C_2 Horizon, Layer 7, (4)	95	94	90	70	28	7	51	29	17	1.7	36	39	A-7, Clay Loam
B & C_1 Horizons Mixture of Layers 3 & 5, (5)	97	93	74	49	21	8	30	10	20	1.7	23	24	A-2-4, Sandy Clay Loam
C Horizon, Layer 6, (6)	100	100	100	80	18	3	38	15	25	1.6	32	30	A-4, Silt Loam

(1), Etc. Location and Number of Soil Sample Taken to Laboratory
Original Ground Line
Grade Line of Roadway

Courtesy Portland Cement Association

Typical Subgrade Survey Sheet for Roadway in Service

NOTE: Subgrade from station 50 + 00 to 69 + 50 reported relatively firm during spring season 19____, except slight softening between stations 56 + 30 and 58 + 00; and stations 67 + 75 and 69 + 75, frost heaving reported winter of 19____ near station 71 + 00 and frost boil reported spring 19____ between stations 69 + 75 and 71 + 75, firm subgrade reported from station 71 + 75 to 76 + 00.

NOTE: Grade line and ground line profiles, stationing, etc., obtained from grading plans. Information given above regarding condition of roadway obtained from soil engineer's notes. Soil survey locates soil types in upper 12 in. of roadway with auger or pick, except where winter and spring condition notes indicate deeper boring should be taken, therefore 3-ft. borings were made at stations 56 + 50, 57 + 25, 68 + 15, 68 + 85, 69 + 50, 70 + 15, 71 + 70, and 72 + 15. Six-foot borings were made at stations 70 + 60 and 71 + 25. As a result of the soil survey, the soil engineer recommends extra thickness of base over sections of roadway in soil layer 7; and 2½ ft. deep excavation, 24 ft. wide, between stations 70 + 00 and 71 + 70, add 50-ft. tapers. If soil cement roadway improvement is to be used, investigate practicability of not placing subgrade treatment (if frost heaving was not severe) and processing soil with cement to 12-in. thickness between stations 70 + 00 and 71 + 70. Also investigate desirability or necessity of processing soil layer 7 to depth of 9 in. or of increasing cement content of this soil in cuts, station 56 + 25 to 58 + 00 and 67 + 75 to 70 + 00. Process balance of roadway to a depth of 6 in.

Fig. 21. Soil profile map showing location of soil types and horizons.

The following scales are only tentative recommendations and are not to be taken as standard. The vertical scale should be exaggerated with respect to the horizontal scale. A vertical scale of 1 inch = 10 feet may work well with a horizontal scale of 1 inch = 200 feet, or a vertical scale of 1 inch = 5 feet with a horizontal scale of 1 inch = 100 feet.

Profiles are taken on the center lines of all runways, taxiways, and aprons.

43. Cross Section. The cross sections for an airport are similar to those taken in highway work but are usually much wider. The general procedure is to take a series of elevations to the right and left of the center line of each runway. These elevations are plotted against the horizontal distances. The paper usually used for this work is 10 x 10 cross-section paper, measuring 10 divisions per inch both vertically and horizontally.

Cross sections at 100-foot stations will usually be sufficient and elevations are taken at each change in ground slope. The following scales are recommended for cross sections: vertical scale, 1 inch = 5 feet; horizontal scale, 1 inch = 5 feet or 1 inch = 10 feet.

44. Grid System. The grid system or leveling over an area may be used to advantage particularly if isometric profiles are to be drawn. The usual procedure is to stake out or mark the field at the corners of a system of squares or rectangles. A 100-foot square makes the best standard figure for airport work and the usual system of numbering may be used. See Fig. 20.

Elevations must be taken at the corners of each square and, since these elevations are to be recorded in a set of level notes, some uniform system of numbering is essential. In Fig. 20 the point x is designated as C_2 and others may be referenced in a similar manner.

A scale of 1 inch = 200 feet is convenient for the horizontal distances on the plan of a grid system.

The elevations are usually marked on the corners of the squares and later the cut and fills are noted. This can be done only after the grades have been established.

45. Soil Surveys. A soil survey includes the examination of soils over an area to a depth sufficient to give the engineer a knowledge of the subsoil conditions which may affect the design of runways, taxiways, buildings, and drainage systems. At the same time ground-water levels should also be carefully noted. Samples of soil must be taken from various depths and this information is then used to plot a soil profile. (See Fig. 21.) All test holes should be located on the base map by a system of surface surveys.

Other classifications of soils will be taken up later but, for the purpose at hand, a few definitions will be given to illustrate the construction of a soil profile.

Soil surveys may be divided into two types: (1) surveys of present runways which are already graded; (2) surveys of new locations which may involve considerable grading.

A soil profile is a vertical section of the soil layers from the surface of the ground downward including the parent material. Soils are defined in three layers or horizons and are designated by A, B, and C. The A horizon is nearest to the surface and may be subdivided into A_0, A_1, A_2. Likewise the B and C horizons may be subdivided into B_0, B_1, B_2 and C_0, C_1, C_2.

Figure 22 illustrates the general system of designation. There is no definite line of division between adjacent horizons as some of the A horizon may filter into the B

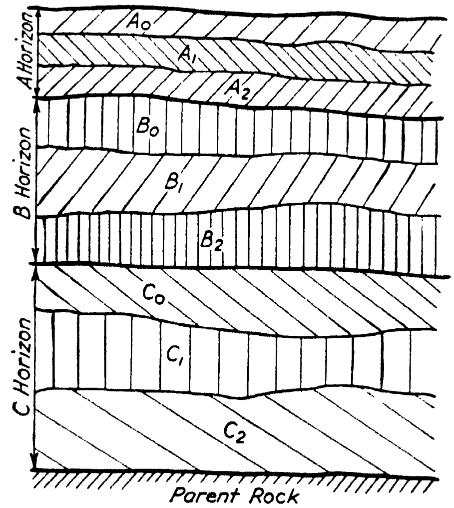


FIG. 22. Soil horizons.

horizon and some of the B horizon into the C horizon and, of course, the reverse may occur. These horizons are deposited by soil-forming processes and are not geological formations.

The terminology used to describe the layers as discovered from samples taken at different levels are given in the following table.

Coarse gravel	Greater than No. 4 sieve or 4.76 mm.
Fine gravel	No. 4 to No. 10 sieve, 2 mm.
Coarse sand	2 mm. to 0.25 mm.
Fine sand	0.25 mm. to 0.05 mm.
Silt	0.05 mm. to 0.005 mm.
Clay	0.005 mm. to 0.001 mm.
Colloids	0.001 mm. to 00 mm. (also included in clay size or fraction).

Depending upon the proportion of each of the above soil separates in the soil mass, soil textures are defined and placed in three main soil textural groups.

1. Soils containing less than 20 per cent clay; soils in this group are called sand, loamy sand, sandy loam, silt loam, and loam.
2. Soils containing 20 to 30 per cent clay; soils in this group are called clay loam, silty clay loam, and sandy clay loam.
3. Soils containing more than 30 per cent clay; soils in this group are called clay, silty clay, and sandy clay.

Chapter V

Grading

46. Establishing the Grade Line. Grade lines are established for many projects such as highways, railroads, pipe lines, and airport runways. It is necessary to have a profile upon which studies may be made and the final location of the grade line on this profile is very important. It has been said that the most economic grade line for a railroad is the one which keeps the cost of operation and the cost of construction at a minimum. This applies to a limited extent to airports.

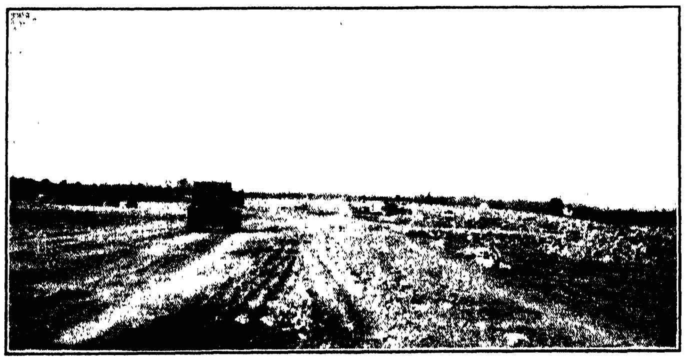
The establishment of the grade line is a matter of trial and frequently trial lines are placed on the profile by the use of black thread and pins. The engineer is thus able to shift the line easily and fix his first estimate for a grade line.

It may be advisable to make changes in this line after some preliminary earthwork computations have been made. The final locations of the grade lines are drawn on the profiles and it is from these that center heights for the cross sections are determined.

It may be said that the fixing of most grade lines on highways and railroads is a two-dimensional proposition but in airports it is one of three dimensions. This is apparent if one thinks of a highway or railroad which involves grading on a relatively narrow strip, perhaps 100 feet wide at the most, and few cross routes. Now consider an airport, where a large area is involved. There may be three or more graded areas with widths of 500 feet or more. These landing areas may cross at varying angles; this means that the grades of one must be made to conform with the grades of two or more other landing areas. In other instances the whole field may be graded. Under these conditions the engineer must visualize the whole grading problem from the viewpoint of length, width, and depth. In a highway, only length and depth are usually involved.

The importance of a careful study of the grades on an airport is evident when one considers the volume of earthwork involved for a slight change in the established elevations. An airport may involve grading over a square mile and one tenth of a foot of earth over this area involves approximately 100,000 cubic yards. If earth moving were figured at $0.40 per cubic yard, the cost would be $40,000.

It can be seen from these statements that a large amount of study in the office may very well be made in establishing the final grades as this cost would be only a fraction of the earth-moving cost.



Courtesy The Asphalt Institute

Fig. 23. An airport is a huge area, involving anywhere from 100,000 to 1,000,000 square yards of paving. The first step is to grade the field to proper cross section.

The procedure in establishing grade lines may be similar to that used on highway and railroad work.

The engineer must have a profile of each runway at the beginning of the study. He then selects one runway, usually the most important, and a grade line is established by trial. This will usually fix the elevation of several points on the other runways. These elevations should be transferred to the profiles of the other runways and used as control points when grade line studies are made on each runway. Several trials may be necessary before a satisfactory combination of grade lines results.

47. Isometric Profiles. Another method for making a study of the grades is to use what are called isometric profiles. The usual method is to determine elevations on the ground at the corners of a system of 100-foot squares. These squares form a grid system which should cover the entire area under consideration.

As illustrated in Fig. 24, consider the square grid system drawn to some convenient scale. (Convenient scales are 1 inch = 100 feet or 1 inch = 200 feet.)

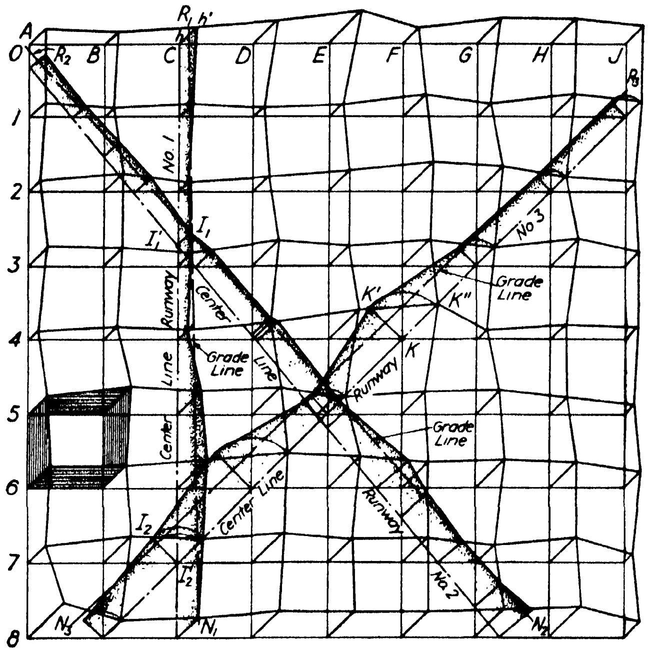


FIG. 24. Isometric profiles.

Imagine this grid system fixed below the field in a horizontal plane and at some definite elevation. It is less confusing if the elevation selected is lower than any point in the field, although this is not absolutely necessary.

1. In Fig. 24 let the grid be placed at elevation 200.00.
2. The elevation of each corner has been determined in the field and the difference between the ground elevations and 200.00 will give the corner height at that point.
3. Draw a 45-degree line from the corner of each square and lay off these corner heights to scale. The scale does not have to be the same scale as used in plotting the grid.
4. Connect these points and you have now constructed an isometric drawing of each prism. (See shaded prism, Fig. 24.)
5. The lines just drawn form profiles from top to bottom or from side to side and may be studied by the usual thread method.
6. Since it is best to follow the natural drainage, it may be advisable to show contours on the plot to indicate the natural slope of the ground. These give the engineer a mental picture of the area and are used for no other purpose.

48. Illustrative Problem. To illustrate the use of these isometric profiles assume that Fig. 24 represents a field upon which we superimpose three runways: runway 1 is parallel to the vertical grid lines; runway 2 extends from upper left to lower right; and runway 3 extends from upper right to lower left. It will be noted that runway 3 has been made to coincide with the 45-degree lines used in the construction of the isometric profiles. This condition may seldom occur but is certainly a possibility.

The datum plane was assumed at the 200-foot elevation so that all elevations are represented to some scale and above the datum plane. The scale used on these 45-degree lines may be anything desired but should not be too small for scaling purposes. A scale of 1 inch = 20 feet is about as small as should be used. It will be noted that, if the datum plane is selected too low or if there is a large difference in the elevations over the area, the diagonal lines may extend over into the next row of squares. This condition is confusing and the remedy is to change the scale on the diagonals or, preferably, use larger squares.

Grade lines may be established by trial, several pieces of black thread and a few pins being used. A fair balance in cuts and fills is attempted by inspection and the grade may be selected to follow the general slope of the whole field. This latter point is one which may help in the design of the surface and subsurface drains. It is always easier to drain with the slope of the topography and uniform depths of drainage ditches will result.

Assume the line R_1N_1 to be a selected grade line for runway 1. Then distance hh' and similar distances will represent the cut from the ground surface to the grade line at C_0.

Grade line 1 and grade line 2 intersect at point I_1, therefore the elevation of the two center lines will be the same point. Since the elevation of this point was established when grade line 1 was drawn, it will be necessary to hold this elevation when grade line 2 is drawn. Likewise the elevation at I_2 was established and must be maintained when the grade line is drawn on runway 3. The elevation at I_3 was established when the grade line of runway 2 was drawn. In other words, grade line 3 must pass through the elevations established at I_2 and I_3.

The profiles and grade lines for runways 1 and 2 are apparent in Fig. 24 but it may be difficult to visualize the profile of runway 3. It will be more convenient if the diagonals are thrown the other way as shown at KK', making $KK' = KK''$. This procedure may be followed throughout for the isometric profile of runway 3. The student should have this latter procedure clearly visualized or it may cause him trouble in using the elevations in combination with elevations of the other two profiles.

49. Using the Cross Sections. The volume of earthwork may be computed by the average end, middle area, or prismoidal method.

1. Average end method:

$$\text{Volume in cubic yards} = \frac{(A_1 + A_2)}{2}\frac{L}{27}$$

2. Middle area method:

$$\text{Volume in cubic yards} = A_m \times \frac{L}{27}$$

3. Prismoidal method:

$$\text{Volume in cubic yards} = \frac{L}{6 \times 27}(A_1 + 4A_m + A_2)$$

In these formulas

A_1 = area in square feet of one end area
A_2 = area in square feet of an adjacent end area
A_m = area in square feet of the middle area (an area halfway between A_1 and A_2)
L = distance in feet between A_1 and A_2

50. Area of Cross Section. The cross-sectional areas are at right angles to the center line of runways and the data for plotting these sections may be taken from a good topographic map, a field survey, or the isometric profiles. Since the cross sections are usually 100 feet apart the value of L in the volume equations is 100.

The procedure in determining the areas of cross sections is to plot them on cross-section paper and then determine the area by one of the following methods:

(a) Planimeter.
(b) Rotometer.
(c) Computations from the dimensions of the section.

The Planimeter. The ordinary polar planimeter may be used by cutting the cross section up into parts so it will be in the range of rotation of the tracing point. It will be remembered that the area traced by a polar planimeter is given by the formula

$$A = hnc$$

when A = the area covered
h = the length of arm
n = the number of revolutions of the wheel
c = the circumference of the wheel.

It is better to have the pole outside of the measured area. If the pole is inside the area it involves finding the area of the zero circumference and to find this one must know the radius of the zero circumference. The work is less involved if the pole is placed outside of the area to be measured.

Some polar planimeters have fixed arms and others have adjustable arms. In the fixed-arm type $hc = 10$ in the above general formula. The area covered in this case will be given by the following formula:

$$A = 10n$$

where A = the area in square inches
n = the number of revolutions taken from the wheel.

If the scale of the map or drawing is 1 inch = 100 feet, then the area A (in square inches) must be multiplied by (100 × 100) or the scale squared. This gives the equivalent number of square feet on the ground.

In plotted cross sections, two different scales may be used such as a horizontal scale of 1 inch = 100 feet and a vertical scale of 1 inch = 10 feet. The area A (in square inches) must then be multiplied by (10 × 100) to obtain the equivalent number of square feet.

The Rotometer. The rotometer illustrated in Fig. 25 consists of a series of concentric circles printed photographically on a transparent low shrink base. A hole at the center D of these concentric circles provides a place for a pin which fixes the rotometer to the drawing. The rotometer then may be rotated about this center.

The circles are drawn so that the space between two adjacent circles is 5 square inches. The numbers on the circles are to guide the operator when measuring an area. The circumference of the rotometer is divided into five units and each of these is subdivided into ten equal parts.

Since the circumference of the circle is divided into five units and the area between adjacent circles is 5 square inches one complete revolution of a point on the rotometer past a fixed point on the drawing paper will correspond to 5 square inches. Any part of a revolution will likewise be recorded on the circumferential scale of the rotometer and opposite the point E which is an index mark made by the operator. It may be selected at any point on the drawing.

1. In order to determine any desired area of a plane surface, first place the rotometer so that both inner boundary A and outer boundary B of the area to be measured are approximately halfway between any two adjacent rotometer circles.

2. To simplify scale readings have visual indicator C to the right. Place push-pin firmly through center bearing of rotometer at D. Then indicate by pinprick or pencil mark a zero or starting point immediately outside of the rotometer between digits 0 and 5, at E.

3. Place and hold any sharp-pointed instrument such as a pen, push-pin, or needle lightly at F (the intersection of the rotometer circle and inner boundary of the area to be determined). Rotate the rotometer in a counterclockwise direction until another boundary and circle intersection G is reached. Remove pointed instrument, return to original boundary, place instrument on rotometer circle intersection with line next higher in number H, and repeat procedure. Continue in this manner until entire area has been traversed with rotometer circles.

4. The scale is then read at the zero E or starting point in units, tenths and hundredths and, by interpolation, thousandths of a square inch. Special scales may be used which read directly in square inches, square feet, acres, square miles, etc., depending on the map scale. The standard rotometer is calibrated to read in square inches.

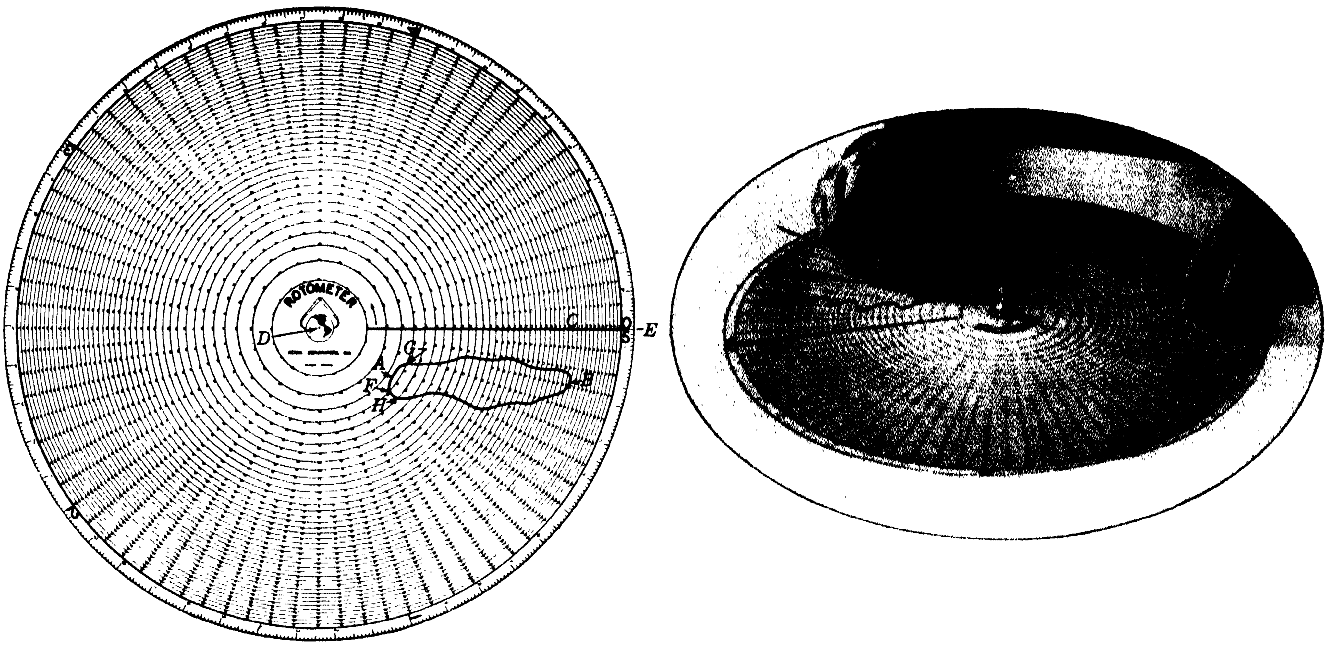


FIG. 25. Rotometer.

The double line extending from the center of the rotometer to the outer scale is used as a visual tally. When the area exceeds one revolution of the rotometer, note should be made of the tally each time this line passes the starting point and the value of each full revolution should be added to the reading of the scale on the edge of the rotometer. One revolution of the standard rotometer is calibrated to read 5 square inches.

Although the same accurate results can be obtained by measuring an area either lengthwise or across, it is somewhat easier to measure with the rotometer placed across the area, especially if a very uneven boundary line exists. See sketch area AB, placed approximately at right angles to rotometer circles.

Computations of Area from Dimensions. The computation may be made from scaled coordinates taken from the plotted cross sections.

FIRST METHOD. The most common method is to use the coordinates in fractional form. If the area of the section shown is to be found, arrange the coordinates in the following form:

$$\frac{x_0}{y_0}\ \frac{x_1}{y_1}\ \frac{x_2}{y_2}\ \frac{x_3}{y_3}\ \frac{x_4}{y_4}\ \frac{x_5}{y_5}\ \frac{x_6}{y_6}\ \frac{x_7}{y_7}\ \frac{x_0}{y_0}$$ (See Fig 26.)

Procedure

1. Multiply the numbers connected by the solid lines and add these products. Notice that the coordinates at the end are the same as at the beginning. This should always be true. This operation may be expressed as an equation as follows:

$$x_0y_1 + x_1y_2 + x_2y_3 + x_3y_4 + x_4y_5 + x_5y_6 + x_6y_7 + x_7y_0$$

2. Now multiply the coordinates connected by the broken lines and add. This operation in equation form is

$$x_1y_0 + x_2y_1 + x_3y_2 + x_4y_3 + x_5y_4 + x_6y_5 + x_7y_6 + x_0y_7$$

3. Subtract these two sums and divide by 2; the result is the area of the section. Since the dimensions are usually in feet, the area will be in square feet. In equation form it will be

$$A \text{ (sq. ft.)} = \tfrac{1}{2}[(x_0y_1 + x_1y_2 + x_2y_3 + x_3y_4 + x_4y_5 + x_5y_6 + x_6y_7 + x_7y_0) - (x_1y_0 + x_2y_1 + x_3y_2 + x_4y_3 + x_5y_4 + x_6y_5 + x_7y_6 + x_0y_7)]$$

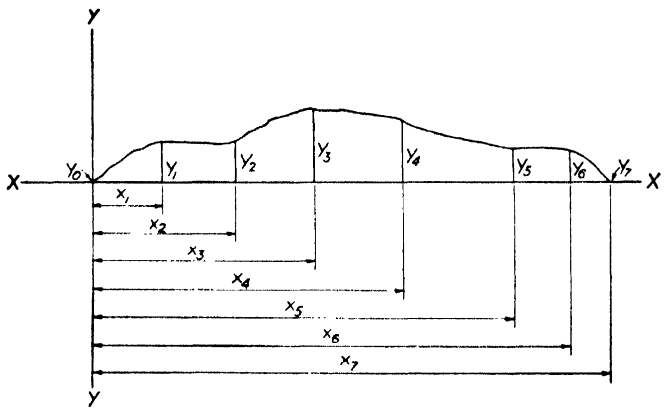


FIG. 26.

SECOND METHOD. The engineer may wish to break up the area into triangles and trapezoids and use simple area formulas for finding the total area. The area of a triangle = ½ base × altitude. The area of a trapezoid = ½ the sum of the bases × altitude.

THIRD METHOD. Another method, sometimes called the stripping method, may be found convenient. The same plotted section may be used and ordinates may be scaled from the drawing at regular intervals throughout the section.

If the area of the section shown in Fig. 27 is to be found, the distance A–B should be divided into a number of equal spaces. If the original surface is irregular, select a greater number of points than if the original surface is uniform. A distance of 10 to 20 feet should be sufficient to give good results.

To illustrate the method of the selection of points let it be assumed that the distance A–B is 618 feet. Then

$$\frac{618}{10} = 61.8, \text{ approximate number of points}$$

and

$$\frac{618}{20} = 30.9, \text{ approximate number of points}$$

This would indicate that the approximate number of points necessary will be between 31 and 62.

Assuming that the original surface is quite uniform let it be decided to use 40 points. Then 618/40 = 15.45 feet, which is the spacing to use for the erection of ordinates on the cross section. It is important that equal spaces be used in this method and this is a point of difference from the first method. In the first method the spacing did not have to be equal.

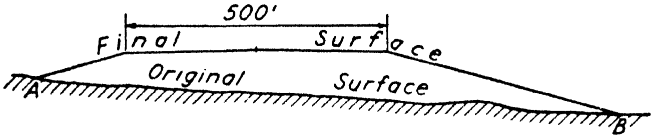


FIG. 27.

Procedure

1. Divide the distance A–B into 40 equal parts and erect ordinates like h_1, h_2, h_3, etc. (Fig. 28).

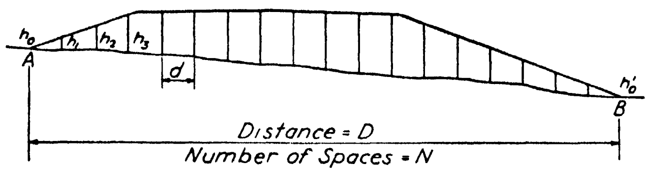


FIG. 28.

2. Scale each vertical ordinate off the drawing.
3. Multiply each vertical ordinate by 2, except the first and last (the first and last ordinates are usually 0) and add them. Call this Σh.
4. Let the distance A–B = D.
5. Then the common distance between ordinates is D/N. Call this d.
6. The area of the section is then

$$\text{Area in square feet} = \tfrac{1}{2}(\Sigma h \times d)$$

where d = the common distance in feet
D = the distance between slope stakes (AB) in feet
N = the number of spaces (one more than the number of ordinates).

Another method which goes under the name of stripping is to scale the middle ordinate of each trapezoid after having plotted the section to scale. The trapezoids referred to are those formed by lines such as h_0, h_1, h_2, etc., in the preceding discussion. The sum of these middle ordinates multiplied by the common distance d gives the area of the section. This common distance is usually taken as 10; this means that only the decimal point needs to be shifted to give the area in square feet.

Chapter VI

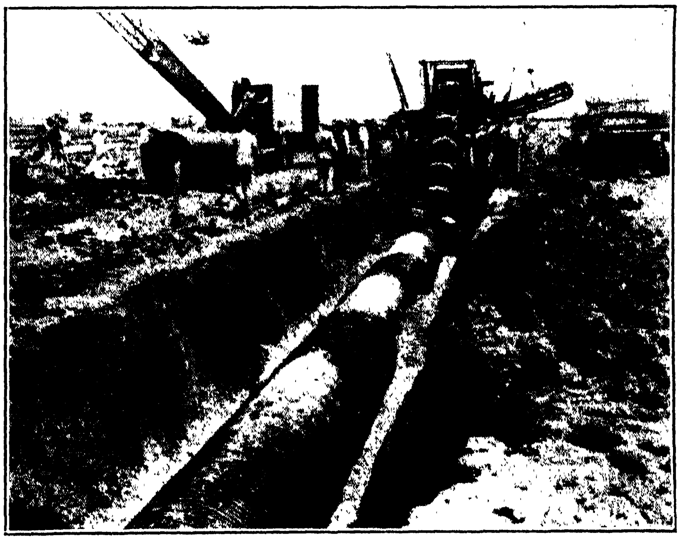
Drainage of Airports

51. Importance of Proper Drainage. A large item in the cost of airport construction never meets the eye of the traveler or visitor to an airport. It is not a factor which will make people exclaim about the attractiveness of the air terminal. Few people other than engineers will realize the existence of an extensive system of drains underlying the landing area for the purpose of maintaining the soil in a stable condition in all kinds of weather. This drainage system also prevents the ponding of water on the landing area to such an extent that the runways become useless for landings or take-offs during or following heavy rainstorms.

The fact that the drainage system does not add as much to the esthetics of the terminal as an expensive and oftentimes useless architectural feature sometimes proves difficult for the engineer when the allocation of funds for the construction is made. When these funds are allocated by a board not made up of technical men it may be difficult to secure necessary funds for this fundamentally important part of the construction, if the sacrifice of an esthetically desirable feature is required. The engineer should be prepared to show the loss in times when the airport may not be usable as a result of improperly drained landing areas. Also, the danger to ships and passengers which exists when attempting a take-off or landing on a poorly drained airport should be emphasized. The added cost of placing drains in at a later time and the need for closing all or part of the airport facilities to do this should be another argument against any tendency to slight the drainage facilities at the time of original construction.

Adequate drainage should produce a landing area where the stability of the soil supporting the loads on runways, taxiways, and aprons would not be dangerously reduced by high moisture content. The facilities for carrying off the water as it falls upon the area in the form of rain should be adequate to prevent ponding of water on the landing area except for such short periods of time as would not materially affect the operation of the landing strips.

The soil stability may be reduced by the accumulation of water either from existing ground water or from surface water entering the ground and increasing the amount of, and raising the level of, the ground-water flow. Moisture may also enter the soil of landing areas by capillary action from the ground water at lower levels. Some soils will become saturated by this method whereas others have little tendency to draw water from below. The capillarity characteristic of the soil is therefore an important item to consider when designing drainage facilities to maintain dry, safe landing areas.



Courtesy Armco Drainage Products Association

FIG. 29. Storm drain during process of construction.

Data secured from the soil surveys should furnish information in regard to the type of soil. This should tell the designer how rapidly water will flow through the soil (its porosity), the tendency of the soil particles themselves to absorb moisture and retain it, the tendency of the soil to raise water by capillary action, and the location of the ground-water stream. With this information the designer

will aim to provide drainage facilities comparable with the soil conditions.

The drainage facilities will be provided to care for water which may enter the subgrade from three separate sources.

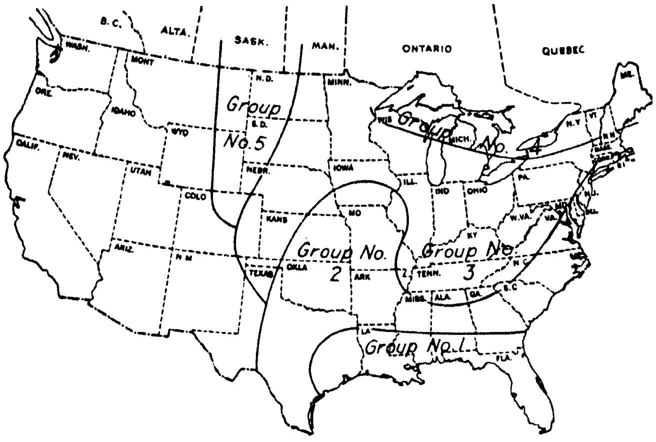


FIG. 30. Rainfall zones. (Adapted from Meyers, *Elements of Hydrology*.)

These are (1) surface water, (2) ground water, (3) capillary water.

The surface water is of major importance since, by preventing it from entering the soil, the amount and elevation of the ground-water flow may be controlled. Capillary water originates in the ground-water stream and by maintaining this stream at a low elevation trouble from capillary water is reduced.

Water falls upon the surface in the form of rain and snow. The amount of precipitation is measured in inches of water over the area. Drainage systems made up of lines of pipe are limited by the rate at which they may carry off the water. The unit of flow used is the second-foot or 1 cubic foot of water passing a given point per second. This indicates the need for considering not only the volume of water but also the rate at which this rainfall is delivered to the area. Rainfall information is available for the various sections of the country; this allows the designer to determine the maximum rate at which rainwater is delivered to the area. This is a result of the rainfall, in inches, and the duration of the storm. The rate is therefore expressed in inches of rain per hour.

The drainage system should theoretically be able to carry off the water at the same rate as that at which it falls upon the area. This is not accomplished in practice. Some amount of ponding in the vicinity of the inlets to the drainage systems is tolerated. The usual criterion in airport practice is to require the drains to carry off the water from a given storm in 1 to 2 hours after the end of the storm.

Rainfalls of short duration are apt to be at a higher rate or intensity than those of longer duration. It is therefore the short storm of high intensity which most seriously taxes the drainage system. Consequently the practice is to design for that maximum intensity of rainfall for a 1-hour period which is not likely to occur more than once each year in the particular area in which the airport is located. This may place the airport temporarily out of service once each year for a short period of time.

The map in Fig. 30 shows areas in the United States where the rainfall intensities are similar. The curves in Fig. 31 show the intensity and duration of the storms which can be expected in the groups of states on the map of the United States in Fig. 30. These show the maximum rainfall which may be expected to be exceeded not more than once each year.

The design rainfall used for an airport located in Ohio, for example, would be that of group 3 as shown on the map and this would be found to be 1 inch for a storm of 1 hour's duration.

During the time from the beginning of a rainstorm until all the water is removed, some water will evaporate and some will be absorbed by the soil. Some surfaces will deliver a greater percentage of the rainfall to the drainage system than others. This quantity of water is known as

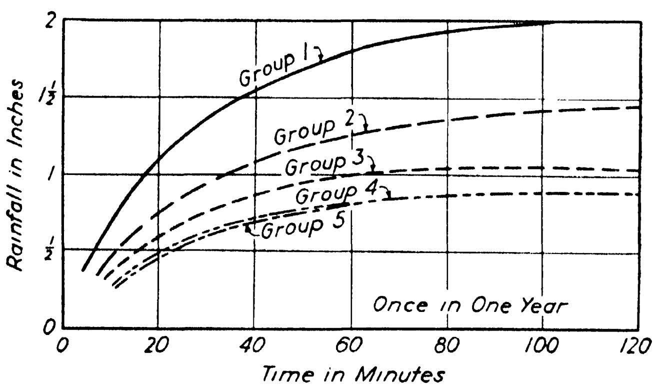


FIG. 31. Rainfall intensities. (From G. W. Pickels, *Drainage and Flood Control Engineering*.)

the runoff and the per cent of total rainfall coming to the drainage system is the runoff factor for the particular area.

TABLE V

SURFACE RUNOFF IN PER CENT

Roof surfaces, and others assumed to be watertight	70–95
Asphalt pavements, properly maintained	85–90
Stone, brick, and wood block pavement, with tightly cemented joints	75–85
Stone, brick, and wood block pavement, with open or uncemented joints	50–70
Inferior block pavements, uncemented joints	40–50
Macadamized roadways	25–60
Gravel roadways and walks	15–30
Unpaved surfaces, no turf or shrubbery	10–30
Parks, gardens, lawns, meadows	15–25
Wooded areas or forest lands	1–20

The selection of the proper runoff factor will materially affect the size of drainage pipes required, as can be seen from Table V.

The variations for any of these types of areas are due to the variation of the slopes of the area and, in unsurfaced areas, the character of the subsoil.

On the basis of the above conditions, an equation may be written expressing the relationship of the quantity of water falling on the area and the pipe capacity of the underdrains.

Q = capacity of pipe necessary, expressed in second-feet
T = the duration of the rainfall in hours
t = time, in hours, allowed for removal of water after storm has stopped
I = rainfall in inches per hour (intensity)
R = per cent of runoff from the type of surface covering the area
A = area of surface drained

Total water carried by pipes = total water running off of area, expressed in cubic feet (from a 1-hour storm)

$$(T + t) \times Q \times 60 \text{ min.} \times 60 \text{ sec.} = A \text{ acres} \times 43{,}560 \times \frac{IR}{12}$$

$$3600(T + t)Q = 3630AIR$$

$$Q = \frac{AIR}{T + t}$$

within precision comparable with the available data used in the equation.

The time interval following a storm during which the surface water is carried away by the intercepting drains will vary with the slopes which carry the surface water to the drains.

(*a*) For average slopes of 0.5 per cent or less the value of t most commonly used is 2 hours.

(*b*) The value of t used for areas where the average slope lies between 0.5 and 1.0 per cent is 1.5 hours.

(*c*) Where the average slopes are greater than 1.0 per cent the time allowed after a storm has ceased is 1 hour.

To illustrate the use of this equation let us use the map and curves in Figs. 30 and 31 for an airport to be located in Oklahoma. This lies in group 2 and the maximum rainfall of 1-hour duration is found to be 1.2 inches per hour.

For 1 acre of turf having a runoff factor of 20 per cent and average slopes of less than 0.5 per cent,

$$Q = \frac{1 \times 0.20 \times 1.2}{1 + 2} = \frac{0.24}{3} = 0.08 \text{ sec.-ft. per acre}$$

For 1 acre paved with concrete and having a runoff factor of 95 per cent,

$$Q = \frac{1 \times 0.95 \times 1.2}{1 + 2} = \frac{1.14}{3} = 0.38 \text{ sec.-ft. per acre}$$

By means of this equation and the rainfall data available in this country a reasonable determination of the capacity required of the drains used to intercept and carry away the surface water may be made.

Where such records of rainfall are not available an estimate of the maximum rainfall to be accommodated by the drainage system will have to be made. This may be done by short time studies of the precipitation and comparison with similar areas where records are available. Parole evidence offered by local inhabitants in regard to the amount and duration of heavy rainfall in past years may serve as a guide in making such an estimate. The natural tendency to exaggerate will undoubtedly affect such reports and this should be considered when attempting to estimate the probable rainfall from such evidence.

In localities subject to rainstorms of high intensity it may be found necessary to increase the ponding allowed by increasing the time allowed after the end of the storm (t) for the complete removal of the storm water by the drains provided. This may be found necessary because of the extremely large drains required to remove the water without serious ponding on the landing area.

Under such circumstances, the major runway might be provided with drainage facilities that would prevent it from being put out of operation while some ponding on the other portions of the landing area might be tolerated.

When ponding on the intervening areas between the runways is resorted to in order to reduce the size of pipe used on runway drains the water is collected at catch basins and carried away by storm sewers. This method must be used when very heavy rainfall is to be expected and where the grades used on sewers and drains must be kept very light because of the naturally flat surface being developed or limitations of the elevation at which the drains may be discharged into natural channels.

The grades used on the ponding areas should be light to minimize the sawtooth effect across the landing field. These light grades require very careful grading to assure flow to the catch basins.

The design of the inlets to storm sewers should be such as to provide a rate of discharge which will not create more ponding of water than can be retained in the area between the runways without flooding the runways themselves. The method of design for these inlets is presented in the U. S. Army Engineers Manual, Chapter XXI, which should be followed when such ponding is resorted to.

52. The Pattern of the Drainage System. The drainage system layout will be a major problem in the design and will be different for each airport site encountered.

The pattern used will be influenced by the type of landing area used. An all-over field must be treated differently than one using a definite system of runways.

An all-over sod or paved landing area has many advantages of operation over the runway system. There are

disadvantages to this type which have contributed to the more prevalent use of runway systems in this country. Cost of paving for smooth, safe landings and the elimination of dust, the maintenance cost of keeping a large area in proper condition, and the higher costs of grading and drainage have been the major disadvantages.

The drainage systems for an all-over landing area must provide efficient drainage to all parts. The drains used throughout the entire area must be strong enough and placed at sufficient depth to withstand the static and impact loads to which they might be subjected.

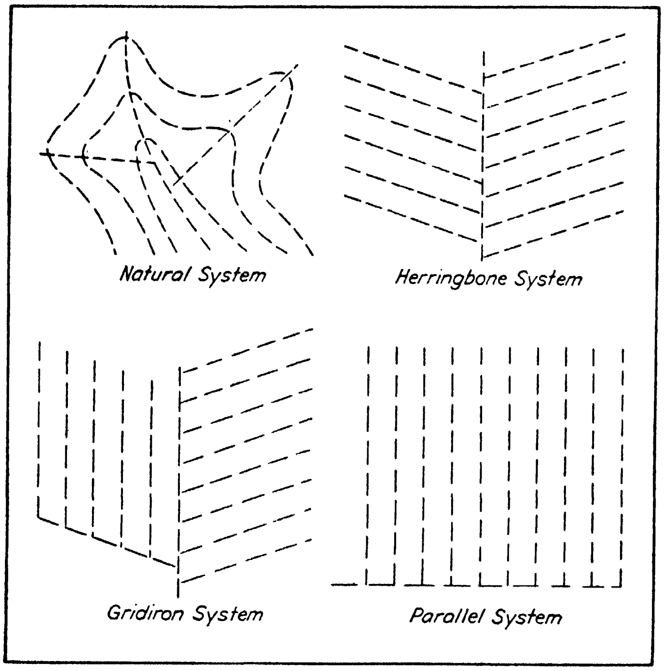


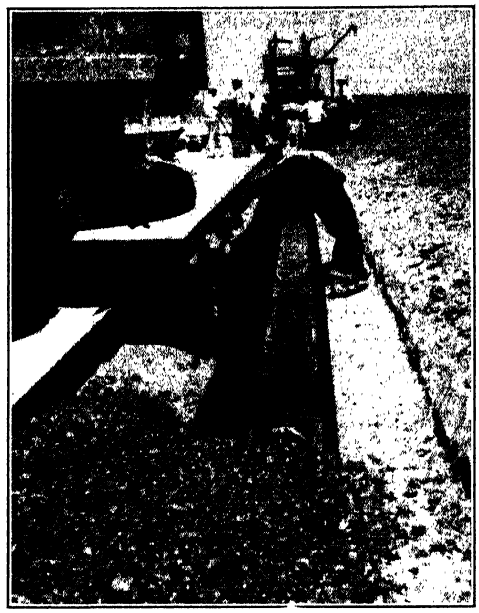
FIG. 32. Types of layouts for subsurface drainage systems as used in all-over fields.

Where runway systems are used a smaller portion of the entire area must be drained to provide support for the plane loads encountered in landing and taxiing operations.

The patterns used will be influenced by the topography of the site. General use of natural slopes will be most economical and grading and planning programs should be coordinated with the drainage systems for the best utilization of such natural slopes.

For all-over landing areas several patterns are in common use. These are illustrated by the sketches in Fig. 32. When a system of runways is used the pattern of drains must follow the runway layout.

Intercepting drains (Fig. 33) placed parallel to the runways are used to collect the surface water. The water is removed from the surface of the runways and adjacent areas by the surface slopes and catch basins or by pervious backfill over the drains.



Courtesy Armco Drainage Products Association

FIG. 33. Intercepting drain.

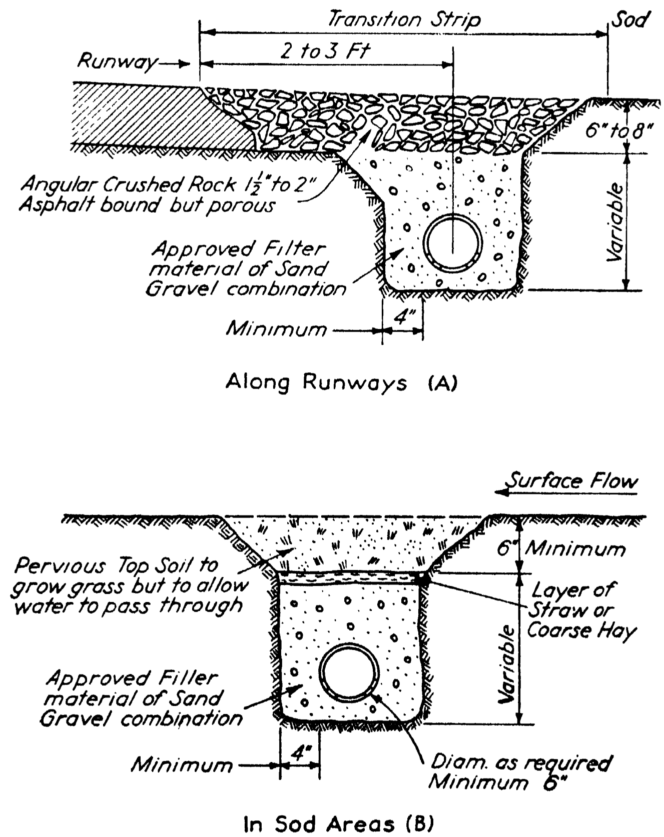


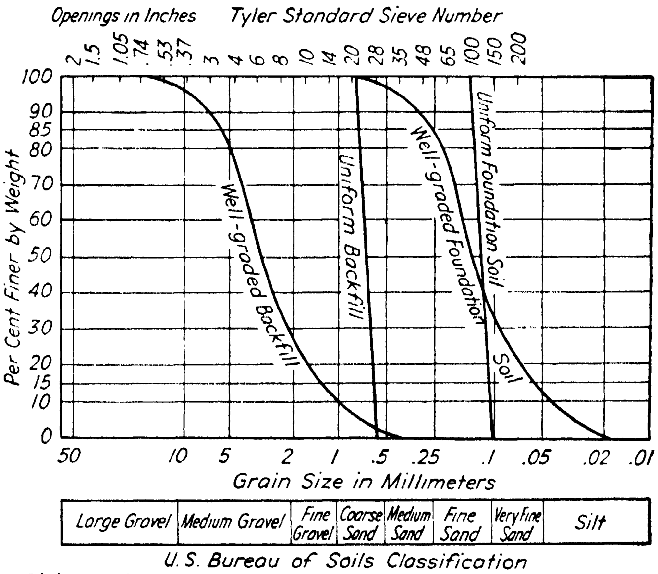
FIG. 34. Type of airport surface intercepting drains recommended for use along runways (A) and in sod areas (B).

The use of pervious backfill directly over the interceptors and adjacent to the runway surface permits the flow of surface water into the interceptors at all points throughout the length of the runway. The pervious backfill may be easily maintained at the same level as the surrounding landing strip, thus offering no obstacle to the safe operation of planes.

The pervious backfill of these runway drains must function as a continuous inlet for the water from the adjoining area. It is essential that these trenches remain pervious and not become clogged with silt washed in from adjoining areas. This silt must also be prevented from entering the drain and clogging it. Therefore, the backfill must function as a filter bed.

Figure 34 shows typical trench construction for these runway drains.

53. Filter Material for Backfilling. The most recent studies made of backfill material were made by the U. S.



Piping Ratio:

$\frac{15\%\text{ Size Uniform Backfill}}{85\%\text{ Size Uniform Foundation}}$, Not Greater than 4.0

$\frac{15\%\text{ Size Well-graded Backfill}}{85\%\text{ Size Well-graded Foundation}}$, Not Greater than 5.0

Permeability Ratio:

$\frac{15\%\text{ Size Uniform Backfill}}{85\%\text{ Size Uniform Foundation}}$, Not Greater than 5.0

$\frac{15\%\text{ Size Well-graded or Uniform Backfill}}{85\%\text{ Size Well-graded Foundation}}$, Not Greater than 25.0

Limiting Gradation for Backfill Surrounding Drain Pipe

Fig. 35.

Waterways Experiment Station at Vicksburg, Mississippi. The report of these tests is contained in "Technical Memorandum No. 183-1," dated November 1, 1941, and revised December 1, 1941.

The greatest difficulty with silting is encountered when the drains must be placed in unstable sand (cohesionless) and coarse silts. The backfill material must prevent these soils from reaching the drain. The coarsest backfill which will accomplish this is the following sand-gravel combination.

Pass a 3/8	100%
Pass Tyler No. 3 sieve	88%
Pass Tyler No. 4 sieve	67%
Pass Tyler No. 6 sieve	50%
Pass Tyler No. 8 sieve	40%
Pass Tyler No. 10 sieve	32%
Pass Tyler No. 14 sieve	25%
Pass Tyler No. 20 sieve	17%
Pass Tyler No. 28 sieve	0

The curves in Fig. 35 give the required relationship of the backfill material to the trench material to give satisfactory filtering and flow to the drain. The piping ratio and permeability ratio shown should be maintained. The actual trench material and backfill material may be analyzed and the curves substituted for those shown. Should the ratios found exceed those recommended, the backfill material should be adjusted to meet those requirements.[1]

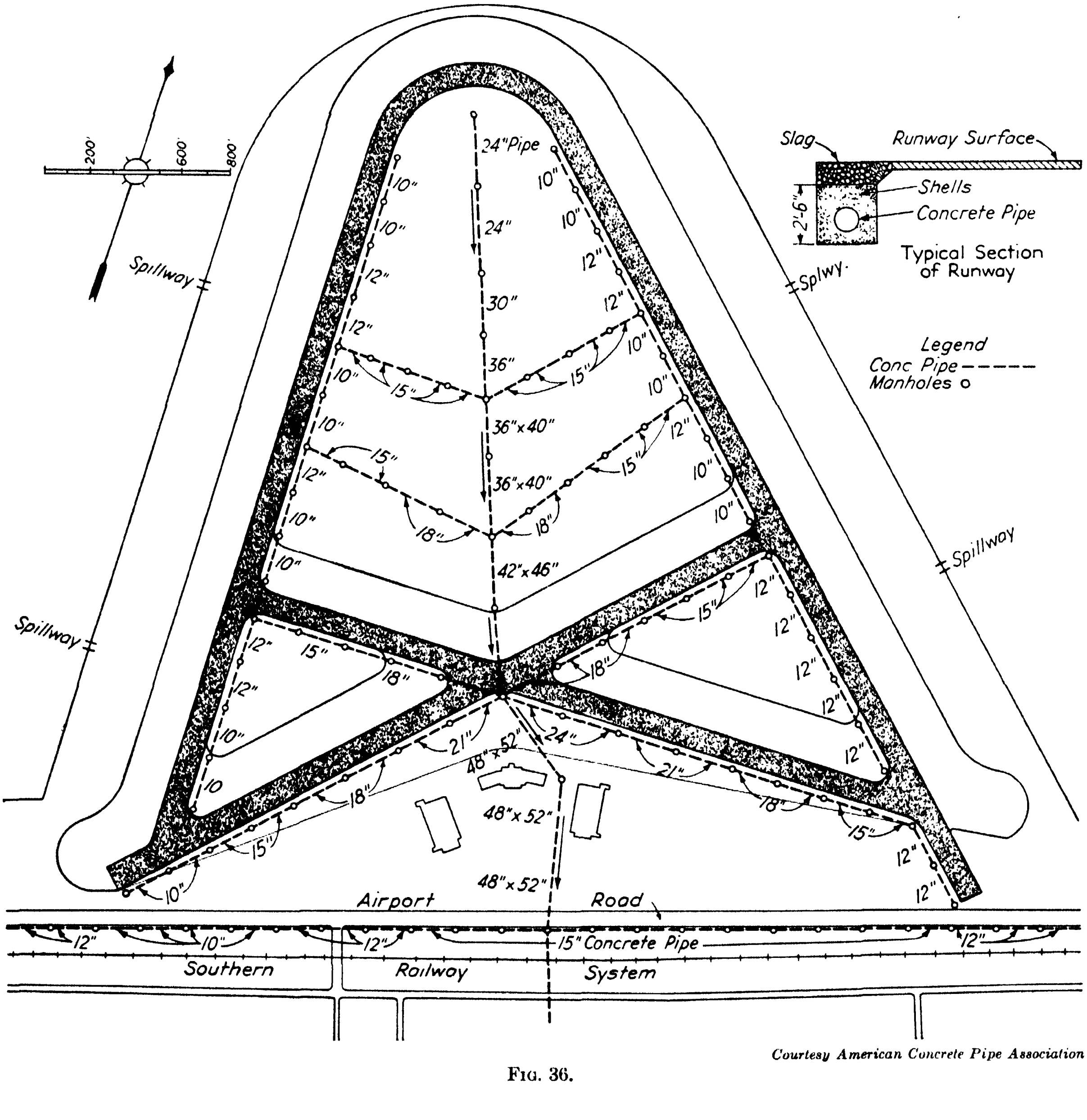
54. Catch Basins to Collect Surface Water. The use of catch basins at intervals of 100, 200, or 300 feet along the edge of the landing area does provide a positive means of access of the surface water to the interceptors. This system has been widely used but owing to the uneven settling of the pavement and catch basin structure, which cannot always be avoided, it is often found that the inlets are left higher than the pavement. This offers a hazard to safe operation and does not serve its function as an inlet for water as originally intended.

In addition to the above objection to catch basins adjoining the runway surfaces, the original grades must be such as to lead the water to these inlets. This requires that the runway surface be a warped surface creating a sawtooth effect along the axis of the landing strips. This is objectionable when planes are landing or taking off.

Surface water from areas other than the runways themselves may be collected by catch basins or by pervious backfill and thence conducted to the main intercepting sewers by lateral drains. Many of these areas thus drained will not be subjected to loads of landing planes and the depth and strength of the drain pipe used need not be as great as would be required in draining an all-over landing area.

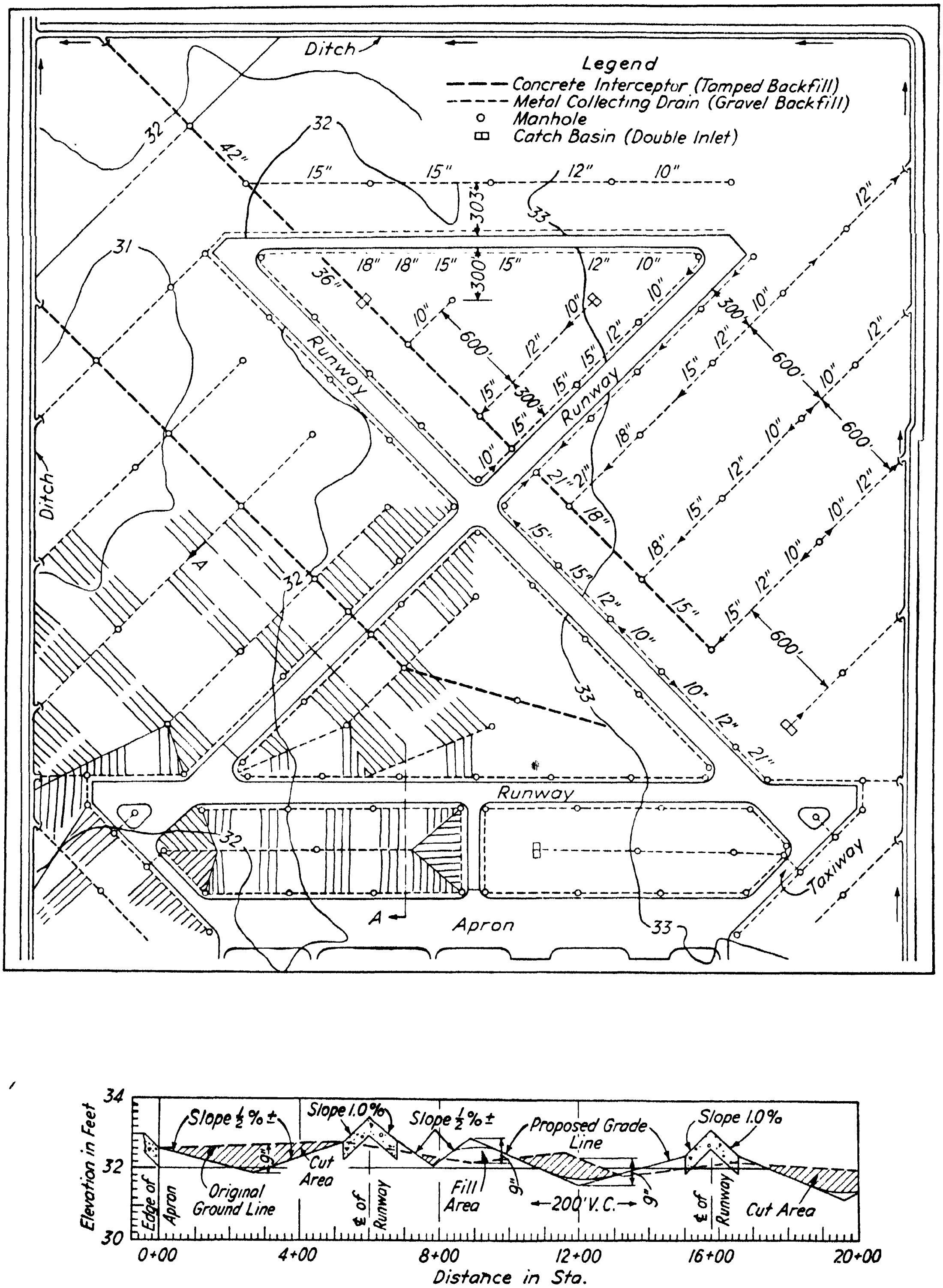
55. Illustrations of Existing Drainage Patterns. On the following pages are shown typical drainage patterns and it will be noted that each is peculiar to the particular site under consideration. Typical sections of runways shown in these drawings show the methods of conducting the

[1] For further discussion of these tests reference may be made to p. 65 in *Engineering Bulletin*, Purdue University, March, 1943, in which Mr. T. B. McClelland discusses filter materials in his paper entitled "Drainage of Highways and Airports."



Courtesy American Concrete Pipe Association

FIG. 36.



Courtesy Engineering News Record

FIG. 37. Drainage system for heavy rainfall conditions.

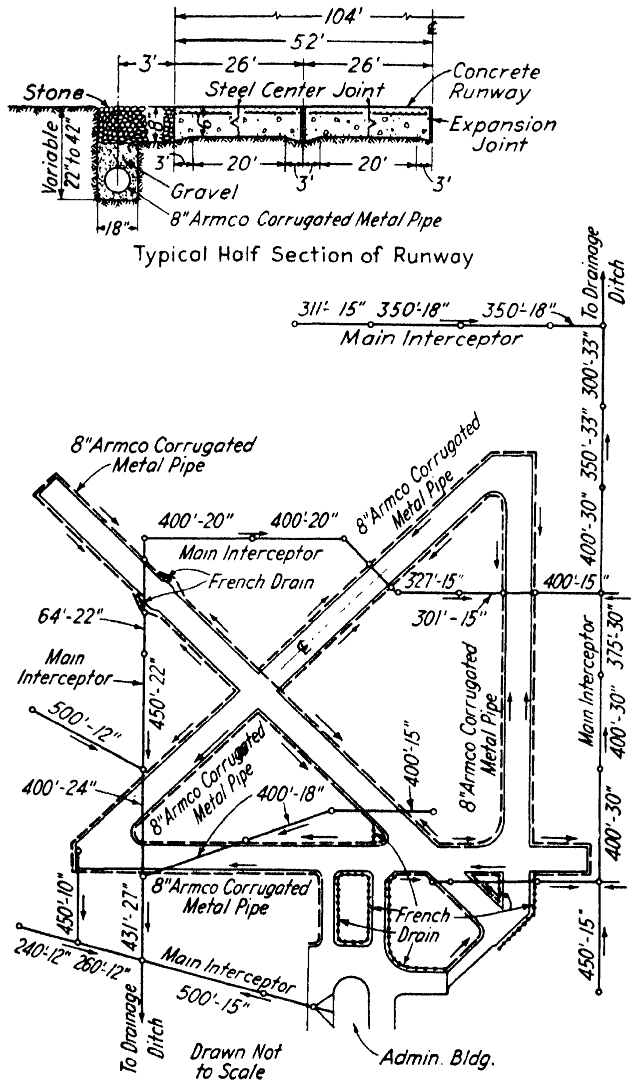
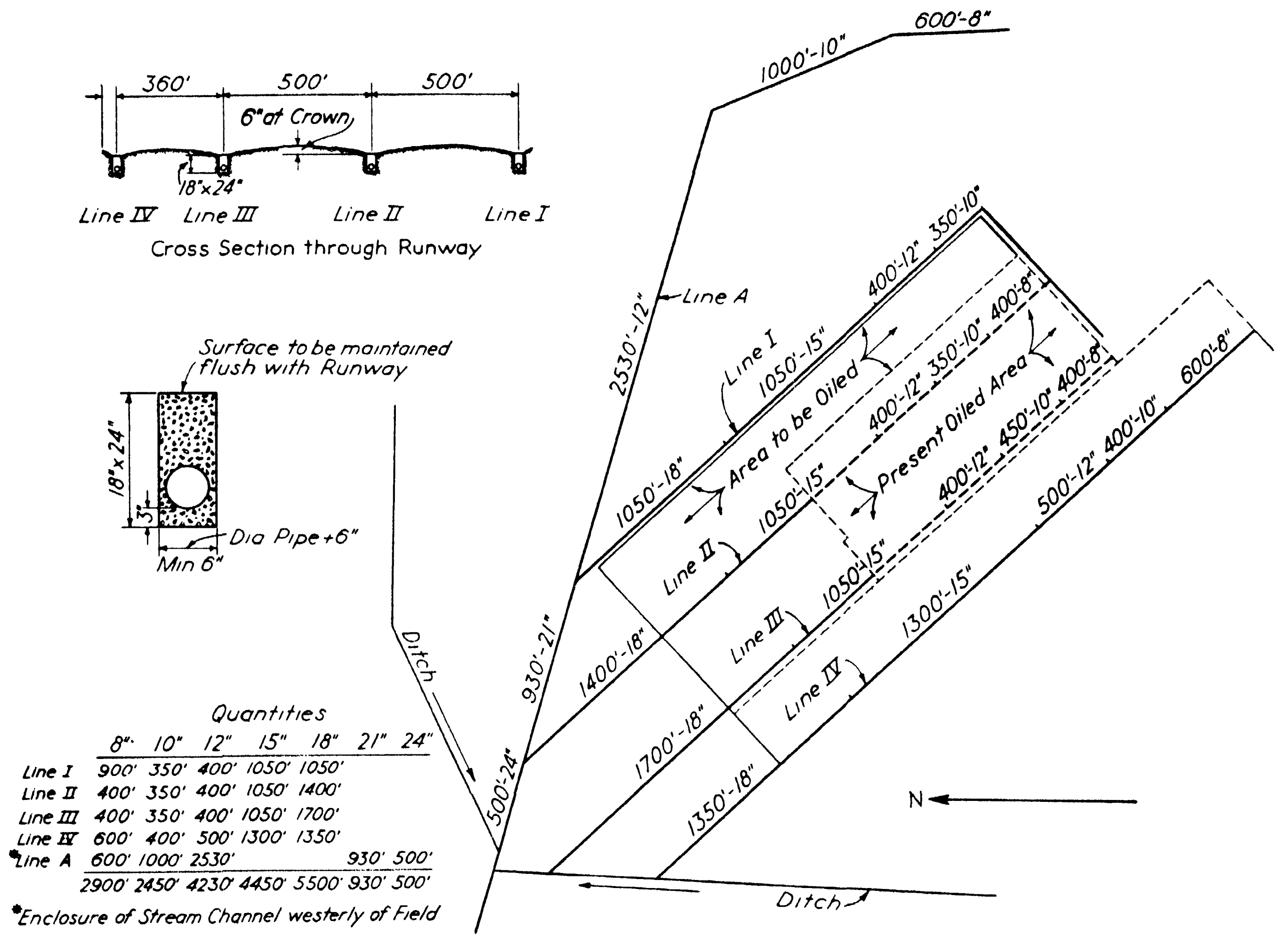

104'
52'
3'
26'
26'
Stone
Steel Center Joint
Concrete Runway
Expansion Joint
Variable 22" to 42"
20'
20'
3'
Gravel
8" Armco Corrugated Metal Pipe
18"
Typical Half Section of Runway
311'-15" 350'-18" 350'-18"
Main Interceptor
To Drainage Ditch
8" Armco Corrugated Metal Pipe
400'-20" 400'-20"
Main Interceptor
French Drain
327'-15"
400'-15"
301'-15"
64'-22"
Main Interceptor
500'-12"
450'-22"
400'-24"
400'-18"
400'-15"
Main Interceptor
450'-10"
431'-27"
240'-12" 260'-12"
Main Interceptor
500'-15"
To Drainage Ditch
Drawn Not to Scale
Admin. Bldg.
300'-33"
350'-33"
400'-30"
375'-30"
400'-30"
400'-30"
450'-15"

FIG. 38.



Quantities

	8"	10"	12"	15"	18"	21"	24"
Line I	900'	350'	400'	1050'	1050'		
Line II	400'	350'	400'	1050'	1400'		
Line III	400'	350'	400'	1050'	1700'		
Line IV	600'	400'	500'	1300'	1350'		
*Line A	600'	1000'	2530'			930'	500'
	2900'	2450'	4230'	4450'	5500'	930'	500'

*Enclosure of Stream Channel westerly of Field

FIG. 39.

water from the surface to the drains at the edge of the runway.

The areas to be drained by each line of pipe should be marked out upon the map showing the drainage pattern. When parallel lines of drains are used, as in draining an area the surface of which is a single plane, the runoff area may be found from the spacing of the lines of drains and the length of the drains. When the area does not all slope in one general direction but is made up of slopes in opposite directions the crown lines must be traced out to determine the drainage area.

56. Illustrative Problem. An illustrative problem is presented here to demonstrate the manner of computing the lengths and sizes of pipes to be used as drains. This is an entirely fictitious problem and is intended as an illustration of the method of computations only.

The sketch in Fig. 40 shows a portion of the runway system of an airport. The assumed location of the air base is in New York State. By using the map and curves of rainfall intensity, a maximum rainfall not exceeded more than once each year is found to be 0.80 inches per hour. (See Figs. 30 and 31.)

There are two types of surfaces involved in this area, namely turf and paved areas. The runoff factor for each of these surfaces will be selected from Table V. The runoff per acre for each class of area may be determined from the equation

$$Q = \frac{AIR}{T + t}$$

in which A = 1.0 acre
I = 0.8 inch per hour
R = 0.90 for paved areas
R = 0.15 for turf areas
T = 1.0 hour
t = 1.5 hours.

For paved areas, Q per acre is

$$Q = \frac{1 \times 0.8 \times 0.9}{2.5} = \frac{0.72}{2.5} = 0.288 \text{ sec.-ft.}$$

For turf areas, Q per acre is

$$Q = \frac{1.0 \times 0.8 \times 0.15}{2.5} = 0.048 \text{ sec.-ft.}$$

For convenience in later computations it is well to compute the runoff reaching the drain in each hundred feet of length.

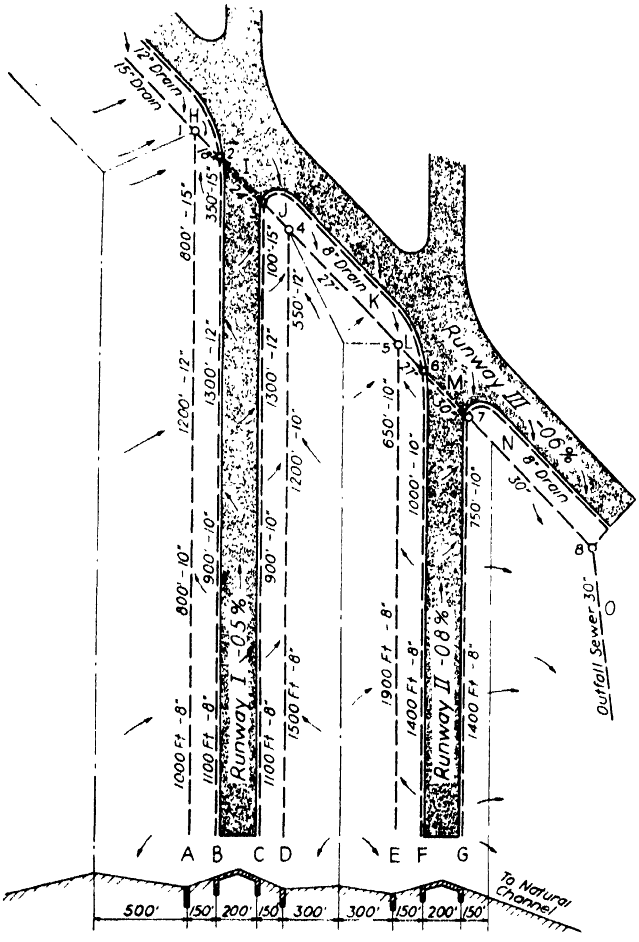


FIG. 40.

The width of the strip of land from which the water drains to each line of pipe may be determined by examining the cross section and the plan given in the problem.

TABULATION OF COMPUTATIONS

Line	Width	Area per Station in Acres	Runoff per Acre	Runoff per Station
A	650	1.492	0.048	0.0716
B, C, F, G	100	0.2295	0.288	0.0662
D, E	450	1.033	0.048	0.0496

The size of pipe necessary to carry any given discharge may be determined by use of Manning's Formula or from diagrams constructed for the solution of this equation. This well-known equation of hydraulics is

$$Q = \frac{A \times 1.486}{n} \times R^{2/3} \times S^{1/2}$$

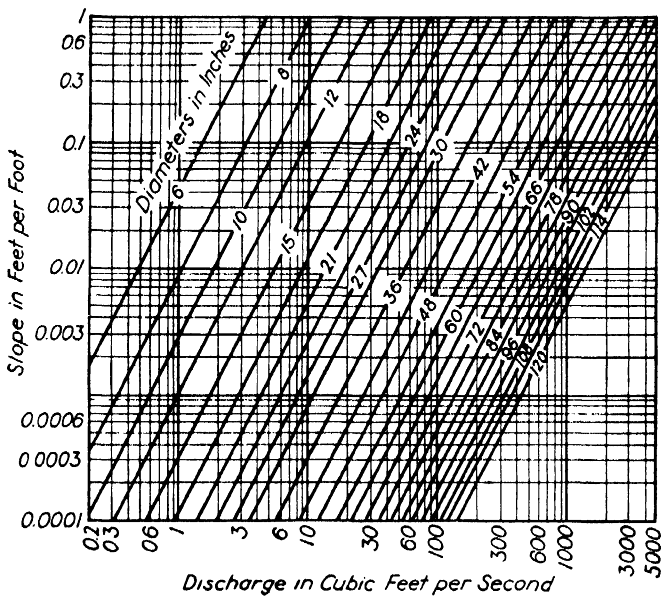
in which A = cross-section area of the drain in square feet

R = mean hydraulic radius which equals the area of the section divided by the wetted perimeter

n = roughness coefficient of the pipe (use n = 0.015)

S = the slope of the drain expressed in feet of vertical rise per foot of horizontal distance.

Diagrams such as that shown in Fig. 41 are available in many hydraulic handbooks for the solution of this equation graphically. This diagram shown is made up for pipe



$n = 0.015$

FIG. 41. Diagram for solution of Manning's Formula for the discharge of pipes.

having a roughness coefficient of n = 0.015. It is used by entering with the known slope of the drain on the vertical axis, following the horizontal line along to the pipe size being used, and reading the discharge capacity vertically below on the horizontal axis.

The smallest size of pipe which will be used in airport design is an 8-inch drain. From the diagram the discharge of this size of pipe laid on a 0.5 per cent grade would be Q = 0.75 second-foot.

The length along each line to the point where the runoff equals this capacity may be determined by dividing the discharge of the pipe by the runoff from the area per 100 feet.

$$L = \frac{0.75}{0.0716} = 10.5 \text{ stations}$$

The discharge and length of each size of pipe used may be found in a like manner and a tabulation of these computations follows.

SIZE AND LENGTH OF DRAINS

Line	% Grade	Pipe Size (in.)	Discharge from Diagram (sec.-ft.)	Runoff per 100 Ft.	Dist. in Sta. to Reach This Capacity	Length Used of Pipe of This Size (ft.)	Actual Q (sec.-ft.)
A	0.5	8	0.75	0.0716	10.5	1000	0.716
		10	1.35	"	18.9	800	1.289
		12	2.20	"	30.7	1200	2.148
		15	3.90	"	54.5	800	2.722
						3800	
B	0.5	8	0.75	0.0662	11.3	1100	0.728
		10	1.35	"	20.4	900	1.322
		12	2.20	"	33.2	1300	2.182
		15	3.90	"	58.9	350	2.415
						3650	
C	0.5	8		Same as *B*		1100	0.728
		10		" " "		900	1.322
		12		" " "		1300	2.182
		15		" " "		100	2.250
						3400	
D	0.5	8	0.75	0.0496	15.1	1500	0.744
		10	1.35	"	27.2	1200	1.339
		12	2.20	"	44.1	550	1.612
						3250	
E	0.8	8	0.95	0.0496	19.18	1900	0.942
		10	1.70	"	34.20	650	1.264
						2550	
F	0.8	8	0.95	0.0662	14.4	1400	0.928
		10	1.70	"	25.7	1000	1.595
						2400	
G	0.8	8	0.95	0.0662	14.4	1400	0.928
		10	1.70	"	25.7	750	1.425
						2150	

The lines *H*, *I*, *J*, *K*, *L*, *M*, *N*, and *O* all carry the runoff collected by more than one drain. These must be designed to carry the total discharge reaching the junction point.

At point 1 two 15-in. drains join.

One is running at full capacity of	3.90 sec.-ft.
The drain from line *A* carries	2.72 " "
Making a total of	6.62 sec.-ft.

From the diagram, using an 0.8 per cent grade on this line, we find that an 18-in. drain will carry 8.0 sec.-ft.

At point 2 the 18-in. pipe carrying	6.62 sec.-ft.
is met by the 15-in. of line *B* carrying	2.42 " "
and a 12-in. from runway III at capacity of	2.20 " "
The total Q is	11.24 sec.-ft.

Line *I* is to be placed on a 0.6 per cent grade. This requires a 24-in. drain which has a capacity of 15.0 sec.-ft.

At point 3 the 24-in. drain carrying	11.24 sec.-ft.
is met by the 15-in. of line *C* carrying	2.25 " "
Making a total runoff of	13.49 sec.-ft.

The 24-in. may be continued to point 4.

At point 4 the 24-in. carrying	13.49 sec.-ft.
is joined by the 12-in. from line *D* carrying	1.62 " "
Making a total of	15.11 sec.-ft.

There will be runoff entering the drain *K* from the triangular area and from the turf area between the runway and the drain.

This area is approximately 6.2 acres which will contribute another to the runoff carried by the drain *K*.	1.54 sec.-ft.
The total runoff carried to point 5 by line *K* is	16.65 sec.-ft.

This will require the use of a 27-in. drain having a capacity of 20.0 sec.-ft.

At point 5 the line *K*, capacity of	16.65 sec.-ft.
is joined by the line *E* carrying	1.26 " "
Making a total of	17.91 sec.-ft.

The 27-in. pipe will be sufficient for the line *L*.

At point 6 the 27-in. carrying	17.91 sec.-ft.
connects with a 10-in. carrying	1.60 " "
and an 8-in. drain of capacity	0.75 " "
Making a total of	20.26 sec.-ft.

This will require the use of a 30-in. drain having a capacity of 26 sec.-ft. on a 0.6% grade.

At point 7 the 30-in. pipe now carrying	20.26 sec.-ft.
is joined by a 10-in. drain from line *G* carrying	1.42 " "
Making a total of	21.68 sec.-ft.

Line *N* receives the runoff from the turf area between the drain and the runway. This runoff is at the rate of 0.048 sec.-ft. per acre. There are 3.78 acres in this strip of turf.

The runoff will be	0.18 sec.-ft.
combining with the total at point 7 of	21.68 " "
Making a total of	21.86 sec.-ft.

At point 8 the total carried in line N is increased by the discharge of the drain along the edge of runway III.

This is an 8 in. pipe carrying a capacity of	0.80 sec.-ft.
Making a total of	22.66 sec.-ft.

The outfall sewer from point 8 may then be a 30-in. pipe which when laid on at least a 0.6 per cent grade will carry a capacity of 26.0 sec. ft., which gives a favorable margin of safety.

This problem has been presented under the assumption that the water will enter the drains through continuous trenches backfilled with filter material. If catch basins were used at intervals to receive the runoff the total reaching each inlet would be computed successively and the size of pipe used between the inlets would be that required at the catch basin at the upper end of the line of pipe.

57. Subsurface Drainage. The supporting power of the soil underlying a landing area is reduced by the presence of excessive moisture. By removing the water which falls upon the surface as rain through the use of a collecting system of drains much of this excessive moisture is removed at its source. There still remains the subsurface water which may be even more detrimental.

The stream of underground water flowing along impervious strata of soil beneath the airport surface will vary in every instance. Its depth and magnitude must be investigated during preliminary surveys.

An unstable soil condition on the finished landing area may be caused by:

1. A soil of high capillarity raising moisture from the subsurface stream and thus causing saturation of the soil in the subgrade. This may be remedied by lowering the flow of ground water or by applying stabilization methods to the soil of the subgrade to prevent this capillary action for a sufficient depth below the subgrade surface.

2. A naturally high level of the ground water as would exist in areas of very light natural grades adjacent to marshes or bodies of water. This requires the use of methods which would divert the flow from beneath the area or the continuous collection of the water sufficiently far below the surface to lower the water table to a satisfactory level.

3. Grading operations may require cuts of such depth that the flow of ground water may be cut, and thus cause the water to flow from the side slopes adjacent to the cuts. Such conditions require that this flow be intercepted and carried away by open ditches or drains to some satisfactory outlet.

The porosity of the soil is a factor contributing to the efficient underdraining. The more granular soils drain more easily than the fine dense soils containing large amounts of clay. A soil classification in the order of the size of the soil particles follows:

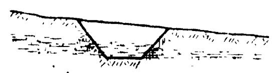
Gravel	Sandy loam	Silt
Sand	Sandy clay	Clay
Medium sand	Loam clay	
Fine sand	Silt loam	
Very fine sand	Clay loam	

The granular group included between gravel and very fine sand drains well but does not compact into a firm surface when dry. Soils with sufficient loam or clay to be compacted to form a good support for surface loads are more apt to retain moisture. The group from sandy loam to clay loam drains well and can be compacted to a condition which will support reasonably heavy loads. The finest soils, silt and clay, are difficult to drain because they retain the moisture and do not readily allow gravitational flow of the free water to drains installed to lower the ground water.

Open ditches, blind drains, tile drains with open joints, concrete pipe with open joints, and metal pipe with perforations may all be used in the control of ground water.

Open Ditch Intercepting Flow Normally Emerging from a Side Hill or Cut Slope

Fig. 42.

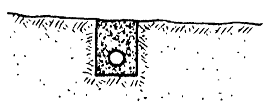


Open Ditch Used to Lower the Ground-water Level

Fig. 43.

A Blind or French Drain, a Trench Filled with Coarse Material of Good Porosity Serves to Lower the Ground-water Level Without the Danger of the Open Ditch

Fig. 44.



A Drain Pipe into which the Water May Enter has the Advantage of a Blind Drain and is More Efficient in Conducting the Water Collected to Some Convenient Outlet

Fig. 45.

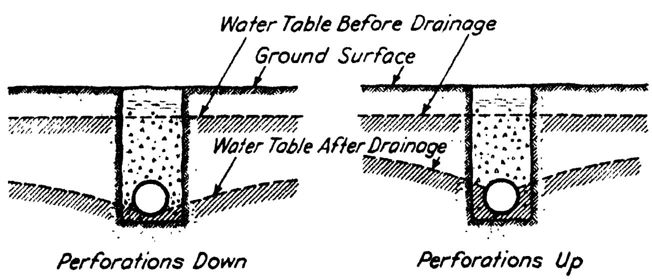


Fig. 46. Comparison to show typical effect on water table when pipe is installed with perforations up or down. (From Armco, *Airport Drainage.*)

Courtesy Armco Drainage Products Association

Fig. 47. Perforated pipe to be used with porous backfill.

58. Depth and Spacing of Underdrains. The depth and spacing of lines of pipe for efficient drainage depend upon the freedom with which water will flow by gravity through the soil particles to the trench and pipe.

Table VI is provided to serve as a guide in selecting the proper spacing of underdrains in order to keep subsurface water sufficiently below the surface of airport landing areas.

TABLE VI

Recommended Depth and Spacing of Subdrains *

Soil Classes	Percentage of Soil Separates			Depth of Bottom of Drain (ft.)	Distance Between Subdrains (ft.)
	Sand	Silt	Clay		
Sand	80-100	0-20	0-20	3-4	150-300
				2-3	100-150
Sandy loam	50-80	0-50	0-20	3-4	100-150
				2-3	85-100
Loam	30-50	30-50	0-20	3-4	85-100
				2-3	75-85
Silt loam	0-50	50-100	0-20	3-4	75-85
				2-3	65-75
Sandy clay loam	50-80	0-30	20-30	3-4	65-75
				2-3	55-65
Clay loam	20-50	20-50	20-30	3-4	55-65
				2-3	45-55
Silty clay loam	0-30	50-80	20-30	3-4	45-55
				2-3	40-45
Sandy clay	50-70	0-20	30-50	3-4	40-45
				2-3	35-40
Silty clay	0-20	50-70	30-50	3-4	35-40
				2-3	30-35
Clay	0-50	0-50	30-100	3-4	30-35
				2-3	25-30

Above data to be considered rough approximations only.
* By courtesy of Armco Drainage Products Association.

59. Size of Pipe for Underdrains. The capacity necessary in the pipe used to underdrain an area will depend upon the quantity of water to be carried away. Where a spring or underground stream is tapped these factors may be measured in the field.

In large, fairly level areas such as airport landing fields the greater part of the ground water comes from the rainfall over the area seeping into the ground. Without considering the flow from tributary areas the ground water would thus consist of the amount of rainfall which was not carried away by the surface drainage system.

Experiments have shown that subsurface runoff may be expressed as a depth of water over the area carried away in a 24-hour period. The rate of flow will depend upon the ground slopes, the slopes of substrata, and the character of the tributary areas. Experiments made by the Engineering Experiment Station at Iowa State College and at the Agricultural Experiment Station at the University of Minnesota indicate that if 5/16 to 3/8 inch of water over the area drained is carried away in 24 hours by the drainage system, the rate of runoff is satisfactory for average soils. A rate of 1 inch of water in 24 hours may be used where the rainfall is heavy and the soil quite pervious.

In determining the size of pipe necessary this rate of runoff is converted to cubic feet per second and the necessary pipe size is selected by using diagrams for Manning's Formula for the discharge in pipes as was done for surface drainage.

Table VII converts runoff in inches per 24 hours to cubic feet per second for convenience in computation.

TABLE VII

Required Capacity of a Drain (cu. ft./sec.) to Remove Various Depths of Water in 24 Hours (Elliott)*

Depth (in.)		Capacity (cu. ft./sec.)	
Fractions	Decimal	Per acre	Per square mile
1	1.000	0.0420	26.88
15/16	0.938	0.0394	25.20
7/8	0.875	0.0367	23.52
13/16	0.812	0.0341	21.84
3/4	0.750	0.0315	20.16
11/16	0.688	0.0289	18.48
5/8	0.625	0.0262	16.80
9/16	0.562	0.0236	15.12
1/2	0.500	0.0210	13.44
7/16	0.438	0.0184	11.76
3/8	0.375	0.0157	10.08
5/16	0.312	0.0131	8.40
1/4	0.250	0.0105	6.72
3/16	0.188	0.0079	5.04
1/8	0.125	0.0052	3.36
1/16	0.062	0.0026	1.68

* By courtesy of Armco Drainage Products Association.

Problem

An area within the runways of an airport is to be underdrained. The soil is sandy loam and the finished surface grade is 1 per cent. The drains may be conveniently placed at uniform depth and in the direction of the surface slope. The length of drain 500 ft.

What are the depth and spacing?

What is the rate of runoff per acre from this area using a $\frac{7}{8}$-in. depth of water in 24 hours as the runoff?

What size of drains should be used?

From Table VI we might place these at 4-ft. depth, and space them 150 ft. apart.

From Table VII $\frac{7}{8}$ in. in 24 hours is 0.0367 cu. ft. per sec. per acre.

$$\text{Area drained by each line is } \frac{150 \times 500}{43{,}560} = 1.722 \text{ acres.}$$

Discharge per line is $1.722 \times 0.0367 = 0.0632$ sec.-ft. From the diagram of pipe discharge it is seen that a 4-in. pipe will carry 0.2 sec. ft. at this slope. The 4-in. pipe is the smallest commercial size of pipe manufactured and would be more than sufficient for this drainage problem.

60. Use of Combined Drains. The above problem indicates the magnitude of the discharge which may be expected from subsurface drains. Since this is small it is common practice to use the same pipe for both surface and subsurface drainage. It is not expected that these drains will be carrying capacity surface water at the same time that the maximum subsurface water is flowing into the drains. The usual design procedure for surface water is to remove the entire amount in 1 to 2 hours following the rainstorm. Maximum capacity of the drains is probably reached about 45 minutes after the beginning of the rainstorm. Experience shows that the maximum discharge from subsurface water occurs from 4 to 6 hours after a heavy rain. In this way the same drains may be used for both surface and subsurface water without overtaxing the pipe if it has been adequately designed for the surface water capacity.

61. Drainage of Runway Bases. Particular attention should be given the underdrainage of the base beneath the paved surfaces. Rational design of these base drains is impractical. The amount of water leaking through the pavement and the amount carried to the base surface by capillary action will be quite variable. This has long been provided for in highway construction by providing a layer of pervious material beneath the pavement, but it is not satisfactory unless adequate outlets are provided for the water collected in this pervious layer. A herringbone system of underdrains placed in the subgrade and leading to the runway drains is the most positive protection against unstable foundations from this cause.

Courtesy The Asphalt Institute

FIG. 48. The use of stone drains for base drainage.

62. Loads Acting upon Underground Pipes. Pipes placed in trenches to serve as drains are subjected to several types of loads.

1. The dead load of the earth fill above the pipe.
2. The static load of a vehicle at rest above the pipe.
3. The impact load of moving vehicles passing over the drain.
4. The force exerted by frost action upon the pipe.

Extensive experiments have been conducted upon pipes to determine the pressures exerted under various conditions of loading. The conditions existing in the field due to impact loads is so variable that the exact stress existing in pipe resulting from impact is a factor of uncertainty in the design of pipe. Impact factors ranging from 1.29 to 2.0 are in use to determine the live load upon the surface of a runway due to landing aircraft.

The procedure of design is to determine the total load per lineal foot upon the pipe for the depth at which it is to be laid. Then select the pipe whose laboratory strength is sufficient.

The dead load is the weight of the column of earth upon the pipe.

The live load is the static wheel load plus the per cent of the static load allowed for impact.

Table VIII gives the dead load which is encountered on vitrified tile and concrete pipes for various depths of cover. The plane loads used today as static design loads for runway paving range from 60,000 pounds for intermediate airfields to 140,000 pounds for heavy bomber bases. These loads are considered to be distributed equally to two wheels or, in the case of dual wheels, to two main sets of wheels.

To simplify the live load determination the sets of curves in Figs. 49 to 54 give the live load transmitted to pipe at various depths for the three plane weights commonly used in design, 60,000 pounds, 100,000 pounds, and 140,000 pounds. These curves are made up by using an impact factor of 2.0.

63. Laboratory Test for Strength. When testing rigid pipe in the laboratory the strength is determined by supporting the tile either in a bed of sand or upon two supports spaced 1 inch apart for each 12 inches of pipe diameter and applying the load along another edge at the top of the pipe.

TABLE VIII

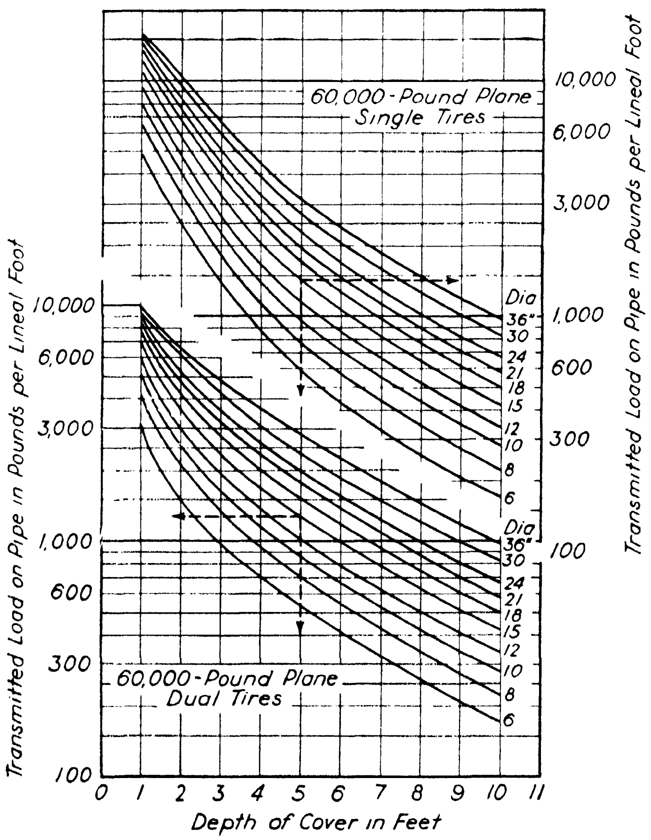
DEAD LOAD ON RIGID PIPE, DITCH CONDITION *

(Pounds per lineal foot)

Diameter (in.)		6	8	10	12	15	18	21	24	30	36	42	48	54
Backfill		Wet Gravel at 120 lb./cu. ft.								Wet Clay at 120 lb./cu. ft.				
Ditch Width (in.)		15	18	21	24	30	33	36	42	54	63	72	81	90
Cover in Feet	1	100	120	150	190	230	270	300	350	420	500	580	670	740
	1½	170	210	250	310	360	410	450	520	640	760	880	1000	1130
	2	230	290	340	400	520	570	630	710	880	1020	1190	1350	1490
	3	310	390	480	560	740	810	910	1070	1370	1590	1810	2050	2280
	4	370	480	590	700	940	1040	1140	1380	1930	2230	2490	2810	3100
	5	420	530	680	820	1100	1230	1350	1680	2340	2800	3220	3620	4000
	6	450	600	750	910	1230	1400	1570	1930	2730	3270	3820	4330	4830
	7	480	640	820	1000	1360	1560	1750	2150	3090	3730	4350	4930	5580
	8	500	680	870	1060	1480	1700	1910	2360	3440	4160	4860	5570	6260
	9	520	700	910	1120	1570	1820	2060	2550	3770	4580	5360	6150	6900
	10	530	730	950	1170	1660	1930	2170	2720	4100	4970	5820	6700	7520

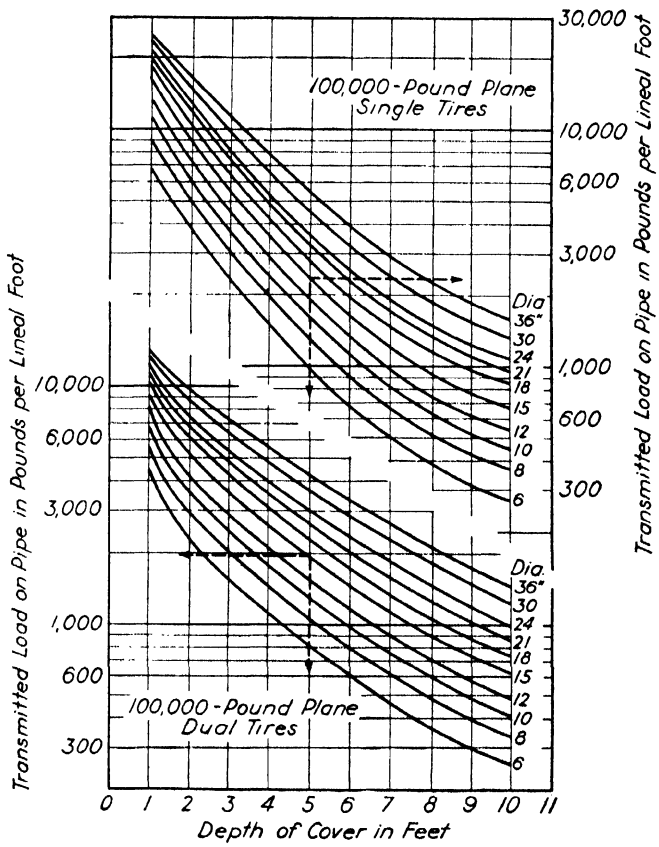
As recommended in Iowa Engineering Experiment Station bulletins, the lesser of the loads computed by the ditch formula and by the projection formula are tabulated. For diameters 12 in. and larger, B_c in the projection formula was considered to be the O.D. of reinforced concrete pipe with standard shell thickness.

* By courtesy of Armco Drainage Products Association.



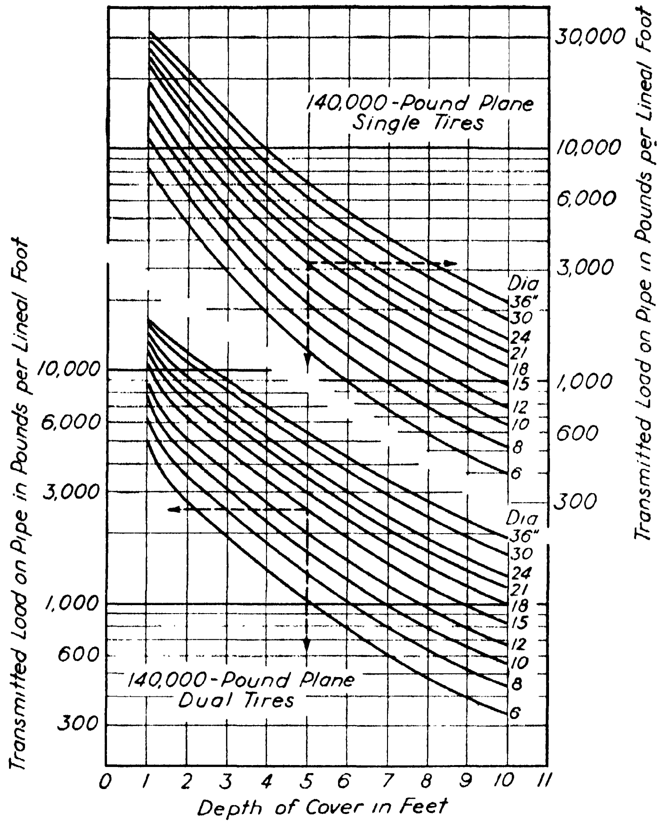
Courtesy Armco Drainage Products Association

FIG. 49. Transmitted live loads to vitrified clay pipe.



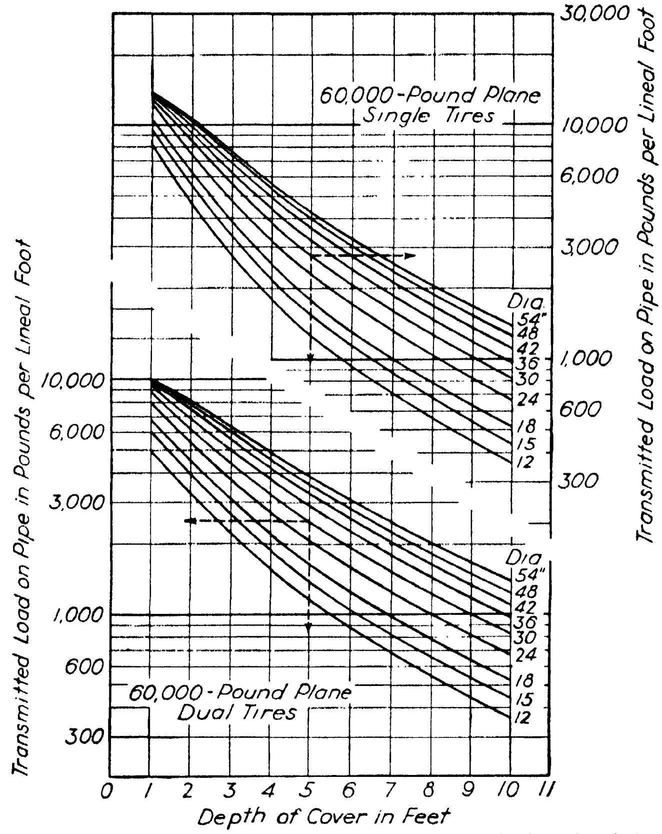
Courtesy Armco Drainage Products Association

FIG. 50. Transmitted live loads to vitrified clay pipe.



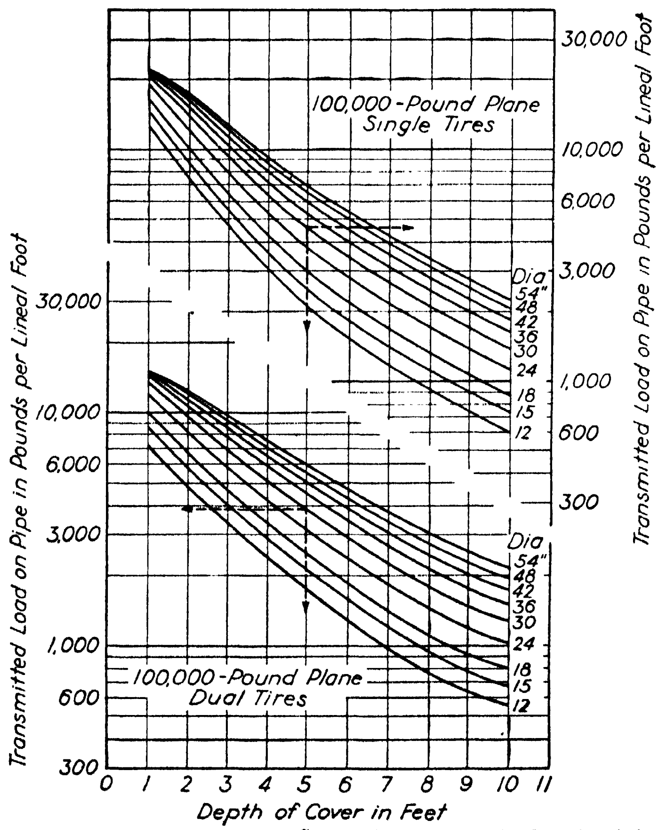
Courtesy Armco Drainage Products Association

FIG. 51. Transmitted live loads to vitrified clay pipe.



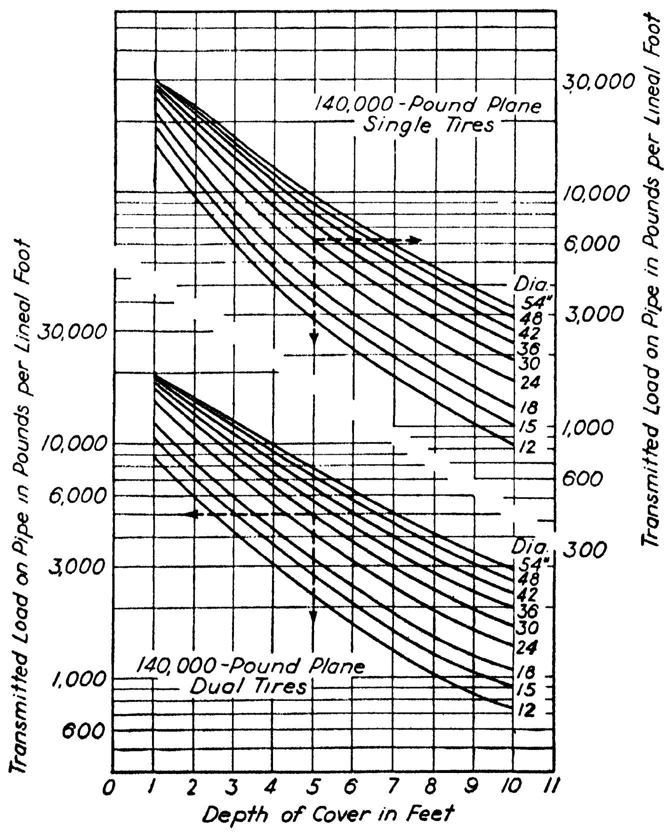
Courtesy Armco Drainage Products Association

FIG. 52. Transmitted live loads to reinforced concrete pipe.



Courtesy Armco Drainage Products Association

FIG. 53. Transmitted live loads to reinforced concrete pipe.



Courtesy Armco Drainage Products Association

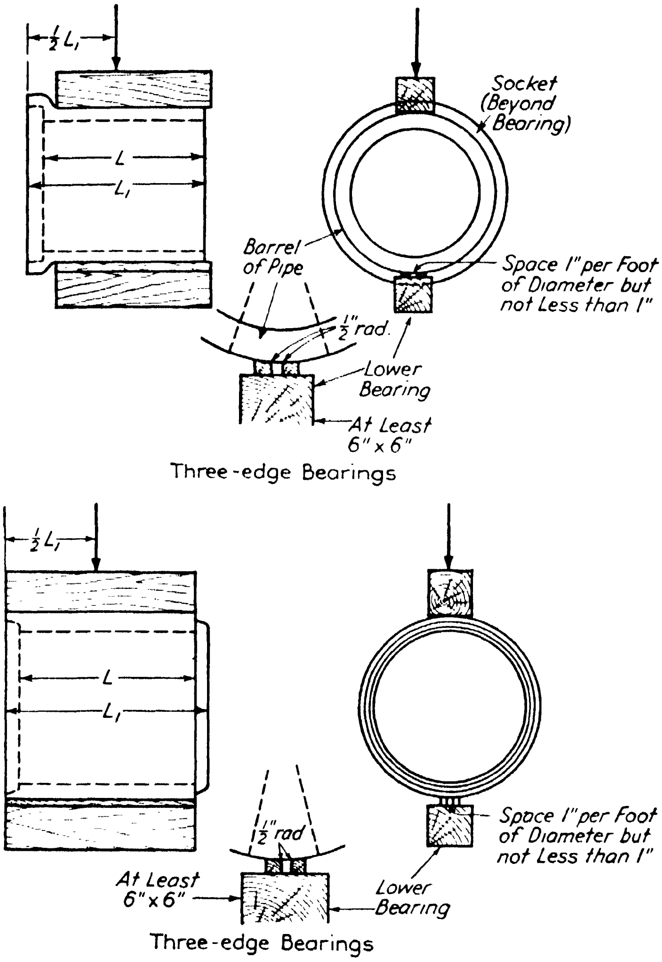
FIG. 54. Transmitted live loads to reinforced concrete pipe.

This latter method is known as the three-edge bearing test and is the one most often used. (See Fig. 55.)

The requirements state that pipe shall withstand a load equal to

$$\frac{(\text{Dead load} + \text{live load}) \times \text{factor of safety}}{\text{Strength ratio}}$$

The factor of safety used for vitrified clay and non-reinforced concrete pipe is 1.5 while 1.25 may be used for reinforced concrete pipe.



Courtesy American Concrete Pipe Association

FIG. 55. Method of supporting rigid pipe for laboratory test.

The strength ratio in this equation is the ratio of the field strength to the laboratory strength. Values used for this are as follows:

Impermissible bedding	1.125
Ordinary bedding	1.5
First class bedding	1.875
Concrete cradle bedding	2.0 to 3.0

Table IX gives values of the laboratory three-edge test which rigid pipe must meet to satisfy the usual specifications for pipe of this kind

Problem

Determine the depth and strength of 8-in. pipe necessary for an all-over field to accommodate a plane weighing 60,000 lb. Frost is found at 3 ft. below the surface; soil is gravel.

Using depth of 4 ft.,

Dead load = 480 lb. per lineal foot (from Table VIII)
Live load = 960 " " " " (from Fig. 49)

Sum 1440 lb. per lineal foot

Safety factor = 1.5
Strength ratio = 1.125

$$\frac{1440 \times 1.5}{1.125} = 1920 \text{ lb. per lineal foot}$$

The strongest 8-in. pipe is the concrete pipe which is required to test 1330 lb. per lineal foot by the U. S. War Department. See Table 9. It will be necessary to place such pipe deeper than 4 ft.

At a depth of 10 ft.,

Dead load = 730 lb. per lineal foot
Live load = 240 " " " "

Total 970 lb. per lineal foot

Safety factor = 1.5
Strength ratio = 1.125

$$\frac{970 \times 1.5}{1.125} = 1293+ \text{ lb. per lineal foot}$$

This is within the test requirements.

To use such pipe under areas where impact loads are encountered it is seen that they must be placed at greater depths than has been the practice in drainage projects where the surface does not carry such concentrated moving loads as are found on an airfield.

Corrugated metal pipe has the property of deflecting under loads and creating more uniform pressures around the circumference of the pipe. It is not subject to collapse as a broken rigid type of pipe would be.

Table X, prepared by a manufacturer of this style of drain pipe, shows the recommended gages for airport drains. According to this table the usual 18-gage 8-inch pipe could be placed just below the frost line without danger of damage from the wheel loads of landing planes.

64. Loads and Depth of Cover for Corrugated Metal Pipe. The curves in Figs. 56 to 65 have been furnished through the courtesy of the Armco Drainage Products Association. These curves furnish data covering a wide range of surface loads and depth of cover. From these diagrams the required laboratory bearing strength may be quickly determined.

65. Selection of the Pipe To Be Used. There are several types of pipe in common use for drainage systems. The purpose for which the pipe is to be used will determine the general type.

For conducting water from one point to another as a sewer the pipe must be a closed line. This may consist of a line of vitrified tile, concrete, reinforced concrete, or corrugated metal pipe. The joints are tightly sealed.

TABLE IX

RIGID PIPE THREE-EDGE BEARING LABORATORY STRENGTH

(Pounds per lineal foot pipe)

Inside Diameter of Pipe (in.)	Drain Tile Vitrified Clay or Concrete			Vitrified Clay Pipe			Non-Reinforced Concrete Pipe		Reinforced Concrete Pipe Load to Produce 0.01-Inch Crack			
	A.S.T.M. C-4-24 *			A.S.T.M. † C-13-40	Federal Spec. SS-P-361 1933	A.A.S.H.O. M65-38	A.S.T.M. ‡ C-14-40T	U. S. War Dept. QMG Div. 1941	A.S.T.M. § C75-40T	A.S.T.M. C76-40T ‖		Federal Spec. SS-P-371 1937
	Farm	Standard	Extra quality	Sewer	Sewer	Culvert	Sewer	2000 D underdrains	Sewer	Culvert standard strength	Culvert extra strength	Culvert
6	533	800	1067	1000	1000		1100	1000				
8	533	800	1067	1000	1000		1300	1330				
10	533	800	1067	1100	1100		1400	1670				
12	533	800	1067	1200	1200	2000	1500	2000	1800	2250		
15	666	867	1067	1370	1370	2500	1750	2500	2000	2625		
18		933	1200	1665	1540	3000	2000	3000	2200	3000		
21		1033	1400	1995	1810	3500	2200	3500	2400			2200
24		1133	1600	2400	2150	4000	2400	4000	2400	3000	4000	2400
27		1233	1800	2765	2360	4500			2550			2550
30		1333	2000	3170	2580	5000			2700	3375	5000	2700
33		1433	2200	3535	2750	5500			2850			2850
36		1533	2400	3900	3080	6000			3000	4050	6000	3000
39		1633	2600									
42		1733	2800						3200	4725	7000	3200
48									3400	5400	8000	3400
54									3700	5850	9000	3700
60									4000	6000	9000	4000
66									4250	6300	9500	4250
72									4500	6600	9900	4500
	Note. A.A.S.H.O. Spec. M. 66-38 same as A.S.T.M. C4-24 except farm tile is omitted.			*Note.* U. S. War Dept. QMG Div. (1941) 2000 D culvert pipe same as A.A.S.H.O. M65-38.			*Note.* Federal Spec. SS-P-371 (1937) same as A.S.T.M. C-14-40T.		*Note.* A.A.S.H.O. M41-38 same as A.S.T.M. C76-40T except 66-in. diameter is omitted. *Note.* U. S. War Dept. QMG Div. 1941 specs. same as A.A.S.H.O. M41-38.			

* "In the strength tests, individual tiles of a standard test whose mean strength is satisfactory may fall 25 per cent below the requirement for the average without causing rejection."

† "Failure of 10 per cent or more of the individual specimens tested to develop 75 per cent of the average crushing strength requirements shall be cause for the rejection of the shipment, . . ."

‡ "Failure of individual specimens tested to develop 75 per cent of the average crushing strength requirements shall be cause for rejection of the shipment, . . ."

§ "Should any of the specimens first tested fail to conform to the test requirements, then the manufacturer shall have the right to additional tests of the size or sizes of pipe which have failed."

‖ "Should any of the preliminary test specimens provided fail to meet the test requirements, then the manufacturer will be allowed a retest on two additional specimens for each specimen that failed, and the pipe shall be acceptable only when all of these retest specimens meet the strength requirements."

This table is reproduced by courtesy of Armco Drainage Products Association.

TABLE X

RECOMMENDED GAGES FOR ARMCO CORRUGATED METAL AIRPORT DRAIN PIPE

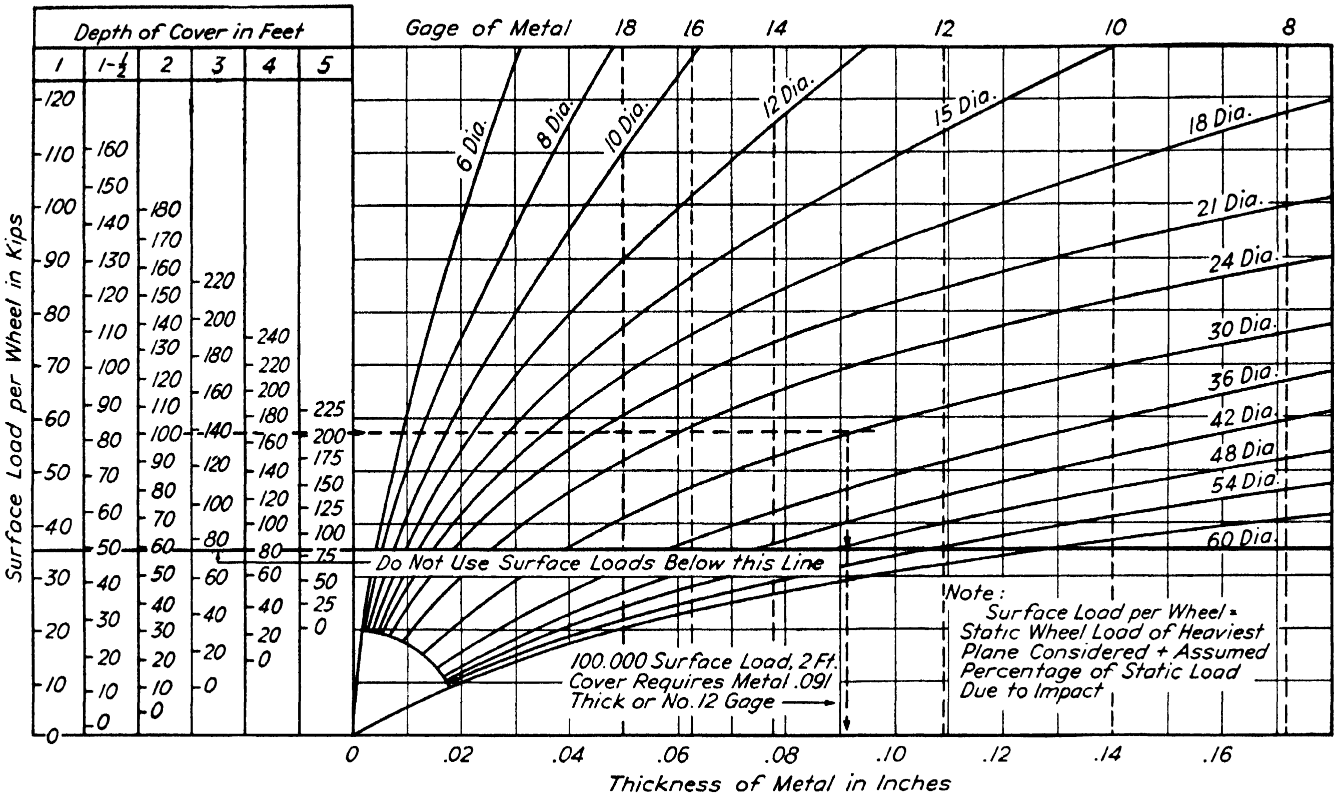
(Impact factor = 2.0)

(60,000-lb. plane)

Diameter (in.)	Single Tires						Dual Tires					
	Height of Cover Over Pipe (ft.)											
	1	1½	2	3	4	5–10	1	1½	2	3	4	5–10
6*	18	18	18	18	18	18	18	18	18	18	18	18
8*	18	18	18	18	18	18	18	18	18	18	18	18
10	16	16	16	16	16	16	16	16	16	16	16	16
12	16	16	16	16	16	16	16	16	16	16	16	16
15	16	16	16	16	16	16	16	16	16	16	16	16
18	16	16	16	16	16	16	16	16	16	16	16	16
21	16	16	16	16	16	16	16	16	16	16	16	16
24	14	16	16	16	16	16	16	16	16	16	16	16
30	12	14	14	14	14	14	14	14	14	14	14	14
36	10	14	14	14	14	14	14	14	14	14	14	14
42	8	12	14	14	14	14	12	14	14	14	14	14
48		10	12	12	12	12	10	12	12	12	12	12
54		8	12	12	12	12	8	12	12	12	12	12
60			10	10	10	10		10	10	10	10	10
100,000-lb. Plane												
6*	18	18	18	18	18	18	18	18	18	18	18	18
8*	16	18	18	18	18	18	18	18	18	18	18	18
10	16	16	16	16	16	16	16	16	16	16	16	16
12	16	16	16	16	16	16	16	16	16	16	16	16
15	14	16	16	16	16	16	16	16	16	16	16	16
18	10	16	16	16	16	16	16	16	16	16	16	16
21	8	12	16	16	16	16	16	16	16	16	16	16
24		12	14	16	16	16	14	16	16	16	16	16
30		8	12	14	14	14	12	14	14	14	14	14
36			10	12	14	14	8	12	14	14	14	14
42			8	12	14	14		10	12	14	14	14
48				10	12	12		8	10	12	12	12
54				8	12	12			8	12	12	12
60					10	10				10	10	10
140,000-lb. Plane												
6*	16	18	18	18	18	18	18	18	18	18	18	18
8*	16	16	18	18	18	18	18	18	18	18	18	18
10	14	16	16	16	16	16	16	16	16	16	16	16
12	12	16	16	16	16	16	16	16	16	16	16	16
15		12	16	16	16	16	16	16	16	16	16	16
18		10	14	16	16	16	16	16	16	16	16	16
21			12	16	16	16	14	16	16	16	16	16
24			10	14	16	16	12	14	16	16	16	16
30				12	14	14	8	12	14	14	14	14
36				8	12	14		8	10	12	14	14
42					10	12			8	12	14	14
48					8	12				10	12	12
54						10				8	10	12
60						8					8	10

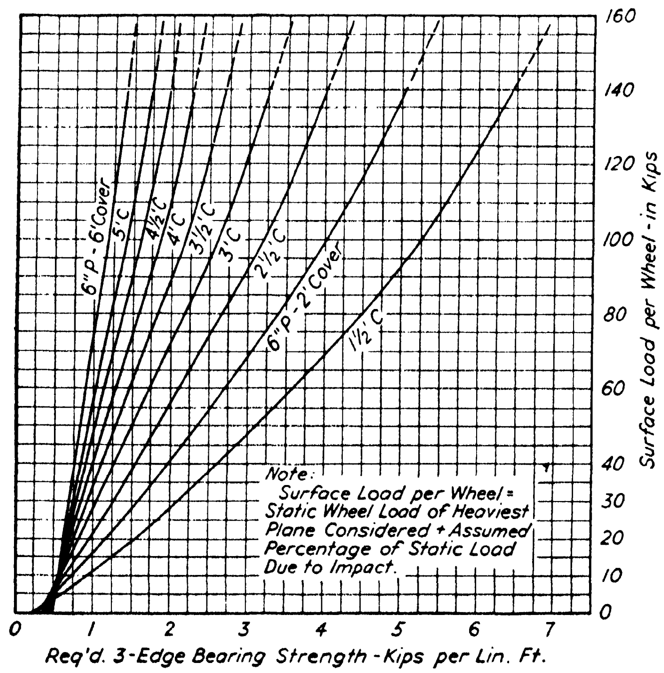
* Gages for 6- and 8-in. diameter based on Hel-Cor-type pipe. All other diameters, standard corrugated. Both types, either perforated or plain.

This table is used by courtesy of Armco Drainage Products Association.



Courtesy Armco Drainage Products Association

FIG. 56. Minimum gages of C.M.P. for various airplane loads and depths of cover.



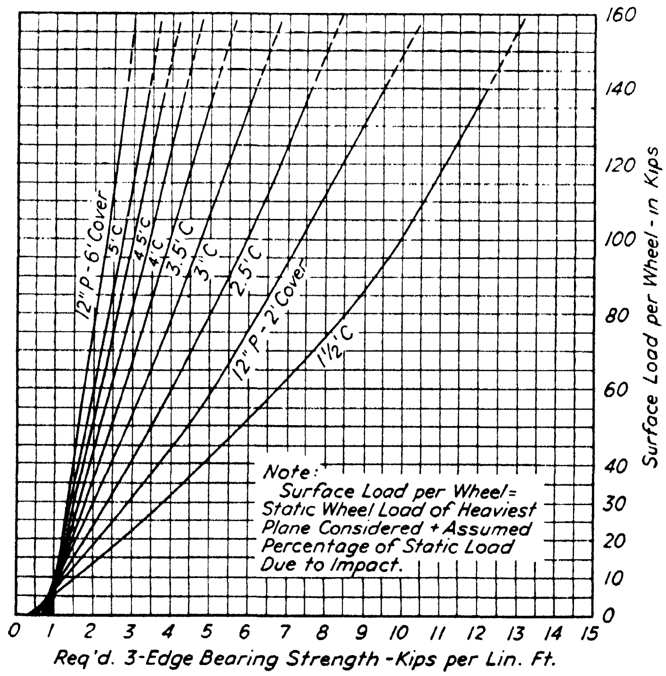
Courtesy Armco Drainage Products Association

6 in. diam. 3.0-ft. sections
Ditch width = 15 in.
Sand-gravel backfill at 120 lb. per cu. ft.

$$\text{Req'd 3-edge bearing strength (lb. per lin. ft.)} = \frac{(\text{D.L.} + \text{L.L.})1.5}{1.5}$$

Live load (L.L.) from single tire per wheel

FIG. 57. Vitrified clay and non-reinforced concrete pipe.



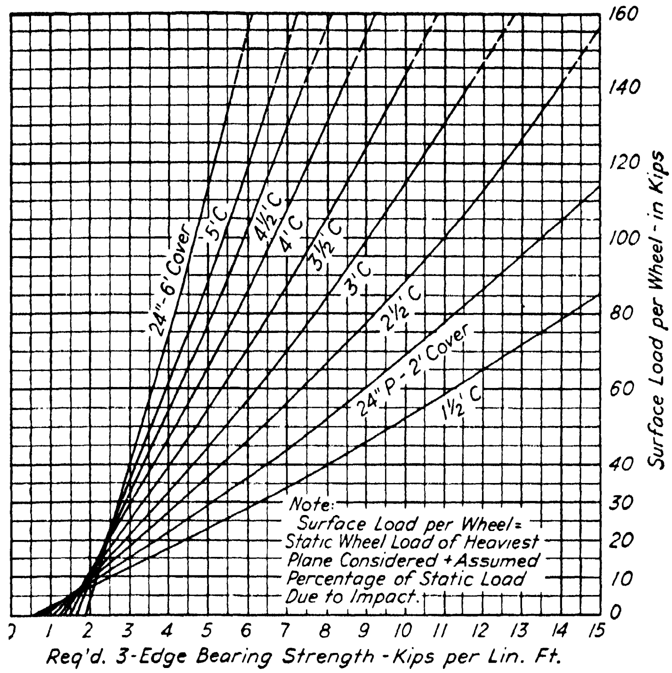
Courtesy Armco Drainage Products Association

12 in. diam. 3.0-ft. sections
Ditch width = 24 in.
Sand-gravel backfill at 120 lb. per cu. ft.

$$\text{Req'd 3-edge bearing strength (lb. per lin. ft.)} = \frac{(\text{D.L.} + \text{L.L.})1.5}{1.5}$$

Live load (L.L.) from single tire per wheel

FIG. 58. Vitrified clay and non-reinforced concrete pipe.



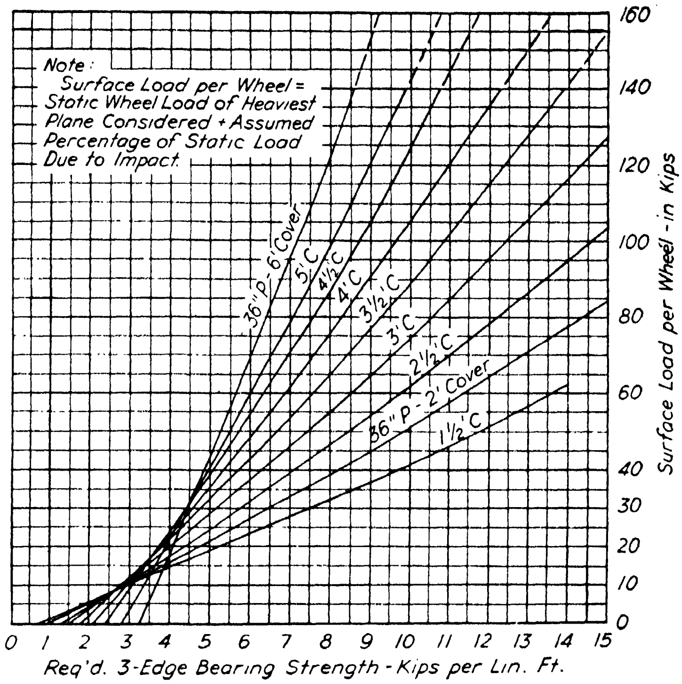
Courtesy Armco Drainage Products Association

24 in. diam. 3.0-ft. sections
Ditch width = 42 in.
Sand-gravel backfill at 120 lb. per cu. ft.

$$\text{Req'd 3-edge bearing strength (lb. per lin. ft.)} = \frac{(\text{D.L.} + \text{L.L.})1.5}{1.5}$$

Live load (L.L.) from single tire per wheel

FIG. 59. Vitrified clay and non-reinforced concrete pipe.



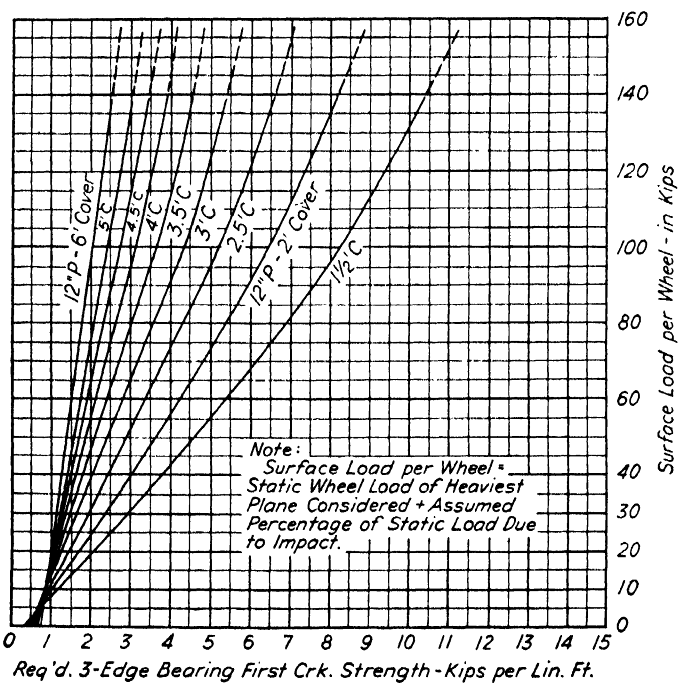
Courtesy Armco Drainage Products Association

36 in. diam. 3.0-ft. sections
Ditch width = 63 in.
Wet clay backfill at 120 lb. per cu. ft.

$$\text{Req'd 3-edge bearing strength (lb. per lin. ft.)} = \frac{(\text{D.L.} + \text{L.L.})1.5}{1.5}$$

Live load (L.L.) from single tire per wheel

FIG. 60. Vitrified clay and non-reinforced concrete pipe.



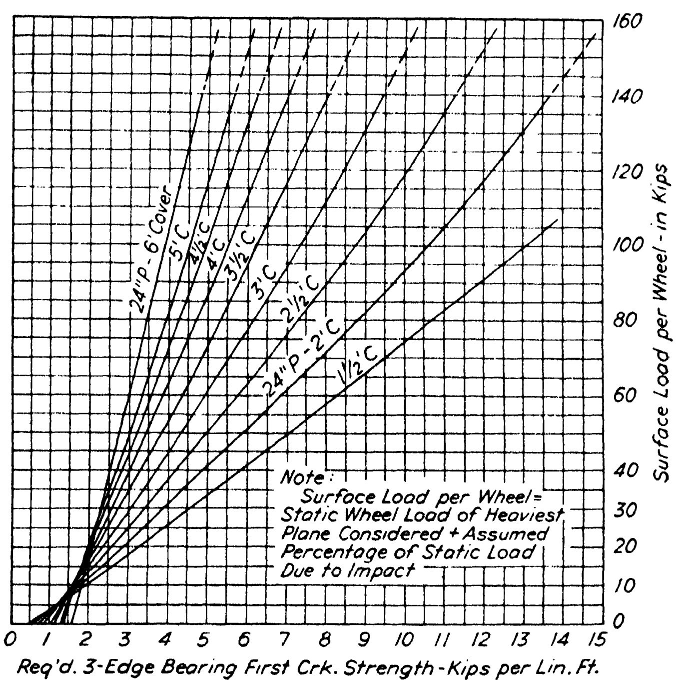
Courtesy Armco Drainage Products Association

12 in. diam. 4.0-ft. sections
Ditch width = 24 in.
Sand-gravel backfill at 120 lb. per cu. ft.

$$\text{Req'd 3-edge bearing first crack strength (lb. per lin. ft.)} = \frac{(\text{D.L.} + \text{L.L.})1.25}{1.5}$$

Live load (L.L.) from single tire per wheel

FIG. 61. Reinforced concrete pipe.



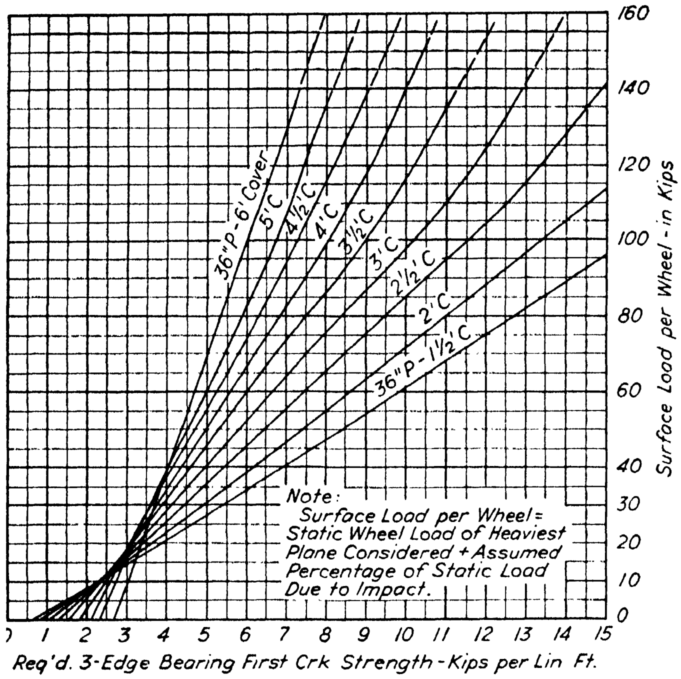
Courtesy Armco Drainage Products Association

24 in. diam. 4.0-ft. sections
Ditch width = 42 in.
Sand-gravel backfill at 120 lb. per cu. ft.

$$\text{Req'd 3-edge bearing first crack strength (lb. per lin. ft.)} = \frac{(\text{D.L.} + \text{L.L.})1.25}{1.5}$$

Live load (L.L.) from single tire per wheel

FIG. 62. Reinforced concrete pipe.



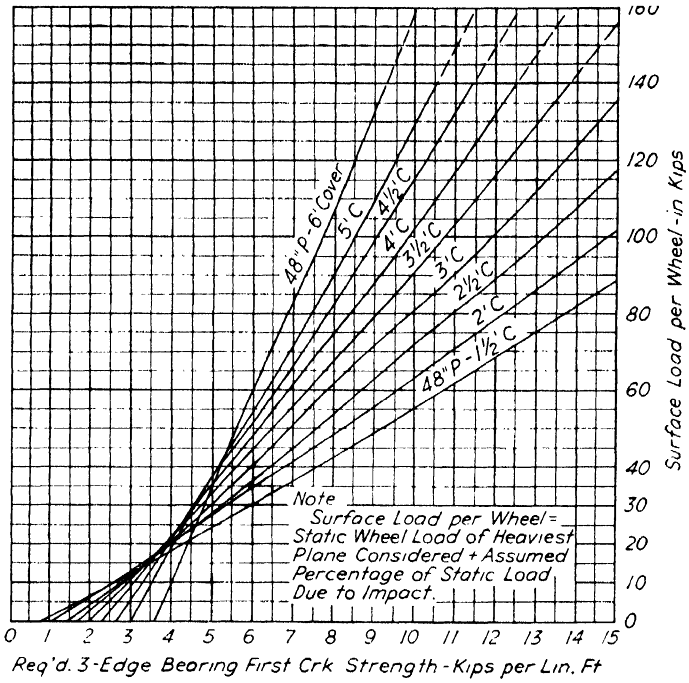
Courtesy Armco Drainage Products Association

36-in. diam. 4.0-ft. sections
Ditch width = 63 in.
Clay backfill at 120 lb. per cu. ft.

$$\text{Req'd 3-edge bearing first crack strength (lb. per lin. ft.)} = \frac{(\text{D.L.} + \text{L.L.})1.25}{1.5}$$

Live load (L.L.) from single tire per wheel

FIG. 63. Reinforced concrete pipe.



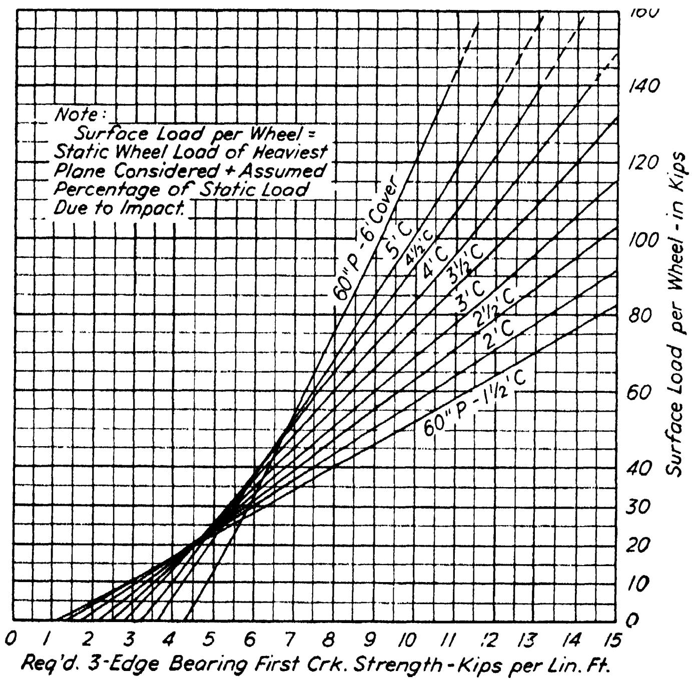
Courtesy Armco Drainage Products Association

48-in. diam. 4.0-ft. sections
Ditch width = 81 in.
Clay backfill at 120 lb. per cu. ft.

$$\text{Req'd 3-edge bearing first crack strength (lb. per lin. ft.)} = \frac{(\text{D.L.} + \text{L.L.})1.25}{1.5}$$

Live load (L.L.) from single tire per wheel

FIG. 64. Reinforced concrete pipe.



Courtesy Armco Drainage Products Association

60-in. diam. 4.0-ft. sections
Ditch width = 96 in.
Clay backfill at 120 lb. per cu. ft.

$$\text{Req'd 3-edge bearing first crack strength (lb. per lin. ft.)} = \frac{(\text{D.L.} + \text{L.L.})1.25}{1.5}$$

Live load (L.L.) from single tire per wheel

FIG. 65. Reinforced concrete pipe.

For surface drainage, subsurface drainage, or combined drains, the pipe may be perforated corrugated metal pipe or tile or concrete pipe with the joints left open to allow the water to enter.

In each of these types, the problem will arise as to which type to select. Some of the characteristics of each will be presented here.

66. Vitrified Clay Pipe. Vitrified clay probably has the longest life when properly laid and not exposed to stresses which will crack the pipe.

Short sections must be laid on a uniform bed to prevent sags from developing in the line. When laid with open joints this pipe may be displaced from line and grade quite easily.

It is adaptable to sewer and drain construction or portions of airfield not exposed to live loads.

When used under conditions with live loads present this pipe must be laid at greater depths than reinforced concrete or corrugated metal pipe.

67. Concrete Pipe. This is a rigid pipe having characteristics similar to the vitrified tile. It has a long life, approaching that of vitrified tile when not disturbed by outside forces.

The available source of supply will be a factor in the selection of either of these pipes. The weight and bulk of these rigid pipes make shipping cost high. The economics of the work would dictate the use of pipe from the source nearest the work.

68. Reinforced Concrete Pipe. This is an excellent pipe of long life and higher strength than previously listed rigid types. It is available in the larger sizes and is well adapted for closed or open pipe lines.

The depths at which such pipe must be laid to resist live loads is slightly less than for non-reinforced rigid pipe. This is due to a lower safety factor being permitted because the reinforcement tends to hold the pipe together even after cracking occurs.

69. Corrugated Metal Pipe. This type of drain is subject to deterioration through rusting unless treated with a bituminous coating. When this is done its life is extended to compare favorably with that of the rigid type of pipe.

The corrugations of the pipe give it the ability to withstand high pressures in all directions.

Its ability to deflect under loads without cracking enables it to withstand higher live load stresses. Although excessive external loads may deform the cross section from a circle to an ellipse, the drain will continue to function. This permits the use of this pipe at depths much less than that required for other types.

Alignment and grade are more easily maintained in difficult trenching because of the longer lengths manufactured. The connections between lengths can be made to produce a continuous pipe.

When used as a surface or subdrain in trenches backfilled with pervious material perforated pipe is used. These perforations are placed in the bottom section and they allow the entrance of the water with less silt being carried into the pipe than when joints of short sectional pipe are left open for this purpose. They also permit the lowering of the ground-water level to a greater depth than if the openings were on the top.

Many purely local considerations will influence the choice of pipe to be used and it will be found advantageous to consider all types for each separate drainage problem encountered.

70. Emphasis upon Importance of Drainage. When designing drainage systems it is essential that no false economies be introduced by inadequate drainage facilities.

Upon efficient drainage depends the efficient operation of the airport. With adequate drainage of the soil underlying the landing area there will seldom occur the necessity of closing the airport because of dangerous conditions of the landing area.

By soil stabilization through adequate drainage a much less costly surface may be used on runways, taxiways, and aprons. This is a major item when one considers the equivalent number of miles of two-lane highway which must be provided for plane movement at a major airport. Equally significant is the fact that the most expensive type of runway surface will fail because of subgrade failure if efficient drainage is not provided.

71. Summary of Drainage Facilities. Figure 66 summarizes the facilities to be installed upon an airport site to

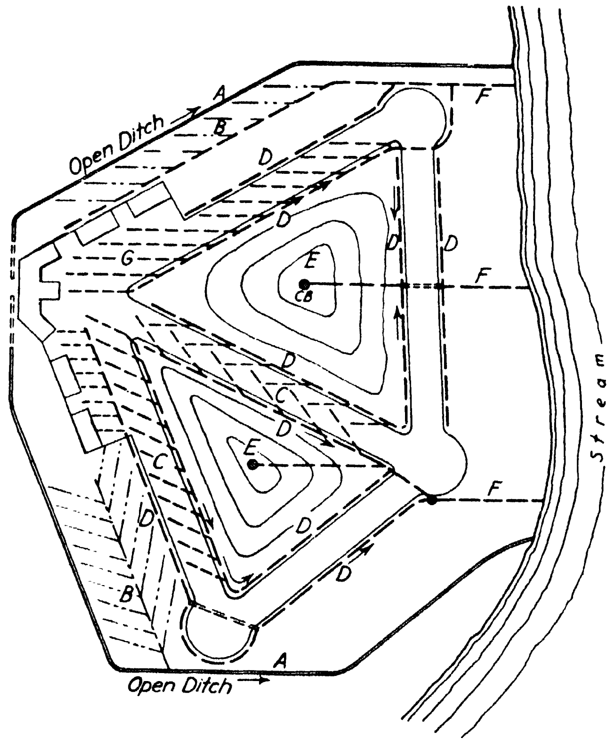


FIG. 66. Sketch showing different types of drainage facilities.

drain the area satisfactorily. The sketch is not to scale but is intended to give a pictorial idea of the drainage system.

A is an open ditch about the boundary of the field for the purpose of intercepting surface flow and underground water before it reaches the landing area.

B represents a system of underdrains laid with open joints or perforations and with the trenches backfilled with a filter material. These assist in lowering the ground water and carrying it away.

C shows patterns of base drains placed in pervious trenches beneath the surface of the paved areas to remove water leaking through from above or rising to the subgrade by capillary action.

D indicates runway drains for rapid collection of the water falling on the runway surface. Their purpose is to remove the water before it floods the landing strips or soaks into the soil and lowers its supporting power. These drains may be laid with sealed joints and they may receive the runoff through catch basins or they may be constructed with open joints or perforations and backfilled with pervious material.

E represents catch basins at the low point of an area where ponding is permitted. The water flows along the surface of the ground to be collected at the inlet and carried away by a storm sewer.

F represents outfall sewers with tight joints used to deliver the water collected by the other facilities to some natural drainage channel beyond the landing area.

Chapter VII

Soil Stabilization

72. Classification of Soils. The common systems for the identification and classification of soils are (1) the system devised by the Bureau of Chemistry and Soils, U. S. Dept. of Agriculture; (2) the system devised by the Public Roads Administration, U. S. Dept. of Agriculture; and (3) the Casagrande soil classification for airfields.

The first system is of minor importance in soil stabilization as it is more concerned with the top layers and the adaptability of the soil to agricultural uses and plant growth.

The second system was designed so that subgrade soils for roadways could be classified and their load-carrying capacities estimated. It may be used for identification of soils irrespective of their proximity to the surface of the earth. The reaction of a soil to both load and climatic changes depends upon five basic physical characteristics. They are cohesion, internal friction, compressibility, elasticity, and capillarity. These physical characteristics control such important performances of subgrade as shrinkage, expansion, frost heave, settlement of fills, sliding of cuts, and lateral flow of soft undersoil. The physical characteristics are furnished by soil constituents easily identified in the laboratory by certain subgrade soil test constants.

The third system was devised by Professor Arthur Casagrande and it is anticipated that experience will lead to some modifications but the present system is considered an improvement over past classifications.

73. Physical Tests. Among the physical test constants which have been suggested as aids in identifying the important subgrade properties are the liquid limit, the plastic limit, the plasticity index, the shrinkage limit, the centrifuge moisture equivalent, the field moisture equivalent, the shrinkage ratio, volumetric change, and the lineal shrinkage. All these tests designate the water-holding capacity of the soil under some defined condition such as the liquid limit where the physical properties of the soil change from semi-solid to a liquid state. In addition to these constants, the grain size of the soil is of considerable importance. The chief value of the soil constants and grain size as a means of identifying subgrade soils lies in the relationship between them rather than in the individual test results considered separately.

74. Grain Size Analysis, A.S.T.M. Designation. The distribution of particle sizes in the soil is determined by a combined sieve and hydrometer method. On the basis of the quantities of soil grains of sand, silt, and clay size the soils are grouped into textural classes, such as sandy loam, silty clay loam, etc. These classes are defined and described in Art. 45 under "Terms Used in Identifying Soils."

75. Physical Test Constants. All the physical test constants are determined on that portion of the soil passing a No. 40 sieve as described under "Preparation of Soil Sample, etc.," A.S.T.M. Designation D421-39. In addition, all test constants are expressed as a percentage of the oven-dry weight of the soil.

76. Liquid Limit, A.S.T.M. Designation. Liquid limit is defined as that moisture content at which the soil would just begin to flow when lightly jarred ten times. It indicates the moisture content at which the soil changes from a semi-solid plastic state to a liquid state.

The liquid limit also indicates the quantity of water required to reduce practically to zero the effects of cohesion and internal friction of the soil. It is, therefore, a rough measure of the activity of the clay particles present in the soil and, in sands, a measure of the shape of the grains and their resistance to flow.

77. Plastic Limit, A.S.T.M. Designation. This constant is defined as the lowest moisture content at which the soil can be rolled into threads 1/8 inch in diameter without the threads breaking into pieces.

The plastic limit equals the moisture content below which the physical properties of the water are no longer identical with those of free water. It is also equal to the critical moisture of cohesive soils, that is, the moisture content at which the bearing power of the soil decreases

rapidly with small increases in moisture content. The prime importance of the plastic limit lies in the fact that it furnishes part of the data required for computing the plasticity index.

78. Plasticity Index, A.S.T.M. Designation. This term is defined as the mathematical difference between the liquid limit and the plastic limit. It is indicative of the increase of moisture content above the plastic limit required to increase the thickness of the water film separating the soil particles to such a degree that the cohesion existing between them is reduced practically to zero. Plasticity indices equal to zero designate non-plastic soils.

At the plastic limit, the soil particles may be considered as having acquired a degree of lubrication sufficiently high to permit them to slide over each other when loaded slightly, although still possessing cohesion in appreciable amounts. At the liquid limit, according to the definition, the soil particles are separated to such an extent that practically no cohesion exists between them. The plasticity index is the difference between these values and may, therefore, be considered a measure of the cohesion possessed by the soil.

Cohesion is provided by the existence of two forces: (1) that furnished by capillary pressure, skin friction, etc., and (2) that furnished by the true cohesion (molecular attraction) of the soil particles.

79. Shrinkage Limit, A.S.T.M. Designation. The shrinkage limit is defined as the moisture content at which a reduction in moisture content will not cause a decrease in the volume of the soil mass, but at which an increase in moisture content will cause an increase in volume of the soil mass. At this moisture content the soil passes from a semi-solid state to a solid state.

The shrinkage limit is an excellent measure of the capillary pressures set up in the soil mass as it dries from a wet state. Low shrinkage limits are accompanied by high shrinkage; this is an indication of extreme activity of the clay portion of the soil. Clay soils having high shrinkage limits are similar to silt soils.

80. Centrifuge Moisture Equivalent, A.S.T.M. Designation. This constant is defined as the moisture content retained by a soil sample which has been first saturated with water and then centrifuged under a force equal to 1000 times gravity for 1 hour.

The centrifuge moisture equivalent distinguishes between soils that are permeable and soils that are not.

81. Field Moisture Equivalent, A.S.T.M. Designation. This term is defined as the minimum moisture content at which a drop of water placed on a smooth surface of the soil will not immediately be absorbed but will instead spread over the surface and give it a shiny appearance.

82. Significance of Grain Size and Physical Test Constants. Charts have been drawn up that show the relationship between the liquid limit and the other physical test constants—the plasticity index, the shrinkage limit, the centrifuge moisture equivalent, and the field moisture equivalent. On the basis of these relationships, the soils have been arranged in groups with respect to their performance when used as subgrades. These groups are A-1 to A-8 inclusive. This is possible because the presence of certain soil constituents indicates the important soil properties. Subgrades may be arranged accordingly.

83. Group A-1. These soils are highly stable under wheel loads, irrespective of moisture conditions. They occur very rarely in a natural state. They have not more than 50 per cent material retained on the No. 10 sieve. The material passing a No. 10 sieve must consist of clay, 5 to 10 per cent; silt, 10 to 20 per cent; total sand, 70 to 85 per cent; and coarse sand, 45 to 60 per cent. The liquid limit is not less than 14 or greater than 25 and the plasticity index is about 3 to 7.

84. Group A-2. These soils contain coarse and fine materials improperly graded or having inferior binder. They are highly stable when thoroughly dry but likely to soften at high water content, caused either by rains or by capillary moisture rise from saturated lower strata. These soils have not less than about 55 per cent sand in the soil mortar (soil mortar equals portion of soil passing No. 10 sieve). Test constants show a liquid limit not generally less than 14 or greater than 35 and a plasticity index of zero to 15.

85. Group A-3. This group includes coarse and fine materials, but with little or no binder. They lack stability under wheel loads but are unaffected by moisture condition. They are not likely to heave because of frost, nor to shrink or expand an appreciable amount with changes in moisture content. The gradation of these soils is such that usually less than 10 per cent of the total sample will have a diameter less than 0.10 mm. The liquid limit is usually under 20 but may go as high as 35, with no plasticity index and no significant shrinkage limit.

86. Group A-4. This group includes silt soils without coarse material and with no appreciable amount of sticky colloidal clay. They have a tendency to absorb water very rapidly in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp they present a firm riding surface which rebounds very little. Soils of this group may cause cracking in rigid pavement as a result of differential frost heaving, and a failure in flexible pavement because of low supporting value. They contain less than 55 per cent sand and have a liquid limit seldom less than 20 or greater than 40. The plasticity index may be as low as zero but it usually is about 5 to 15. Shrinkage limits will usually be less than 25.

87. Group A-5. These soils are graded similarly to soils in Group A-4 but they furnish highly elastic supporting surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper

compaction of macadam during construction and with retention of good bond afterwards. They have a grading of less than 55 per cent sand, a liquid limit usually greater than 35, and a shrinkage limit usually greater than 30.

88. Group A-6. These are clay soils without coarse material. In a stiff plastic state they absorb additional water only if manipulated; they may then change to the liquid state and work up into the interstices of macadam or cause failure as a result of sliding in high fills. These soils must be at a stiff consistency to furnish the firm support essential to properly compacting macadam. Shrinkage properties of these soils which have large amounts of colloidal clay may combine with alternate wetting and drying under field conditions to cause cracking in rigid pavements. Gradation of these soils is seldom less than about 30 per cent clay. The liquid limit is greater than 35, with a relatively high plasticity index. The shrinkage limit is usually low, around 10 to 14.

89. Group A-7. These soils are similar to Group A-6 but at certain moisture contents deform quickly under load and rebound appreciably upon removal of the load as do subgrades of Group A-5. Alternate wetting and drying under field conditions lead to even more detrimental volume change than in subgrades of Group A-6. Grading is seldom less than about 30 per cent clay. The liquid limit is usually greater than 35, and the shrinkage limit is greater than the A-6 soil.

90. Grade A-8. Soils in this group are very soft peat and muck incapable of supporting a road surface without being previously compacted.

91. General. The percentages of clay, silt, and sand are used in defining the different groups. These terms, how-

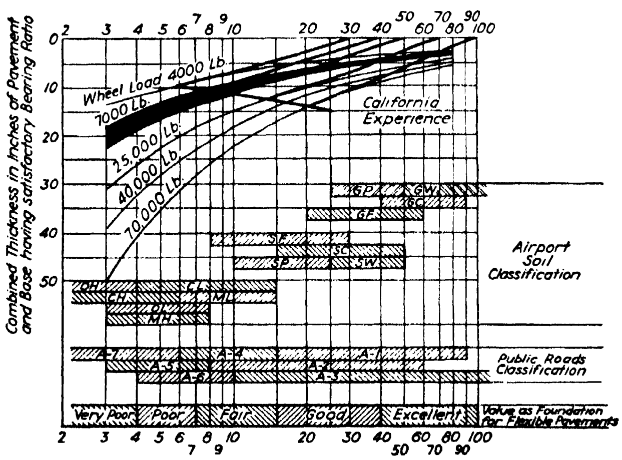


FIG. 67. California bearing ratio at 0.1-inch penetration for compacted and soaked specimen

ever, refer to the physical properties generally assumed to be possessed by these constituents or combined constituents rather than to the definite grain sizes. Therefore, in the identification of members of the different soil groups, grain size is subordinate to physical properties.

Thus, we may have a soil that classifies as a clay from a textural standpoint but if physical test constants are such

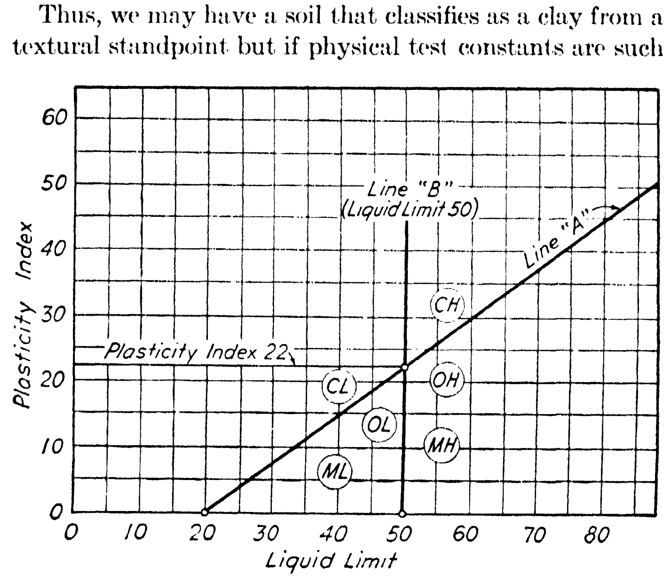


FIG. 68. Liquid limit *versus* plasticity index chart.

that the shrinkage limit is big and the liquid limit is low, it might classify as an A-4 subgrade soil and be similar in its action to regular silt soil.

92. Meteorological Conditions. It is very easy to make a mistake in designing a pavement by simply assuming that such design depends upon soil analysis only. The location of the airport in respect to climate has a profound effect, and a procedure which is entirely satisfactory in New Mexico, for example, may be utterly unsuitable in Maine. In a war period, when an engineer may find himself anywhere, it is very necessary to take these differences into consideration. They are pointed out because most engineers have had their training within a limited area and, when transported to another location, they are very apt to think in terms of the types of soil and pavements with which they are most familiar. Therefore, it cannot be too strongly emphasized that meteorological conditions are all important and that temperature and moisture changes must be given full weight.

For example, an A-7 soil, which is generally thought of as a very poor soil from a road-building standpoint and which under no conditions would support a thin pavement in New York State, might be entirely satisfactory on the plateau of Bolivia. In some plateau areas, or desert areas, there is no rainfall; consequently, there is nothing to produce volume change in the soil even though the temperature range may be considerable. If a little crankcase oil were sprinkled on the surface to keep the dust down, the heaviest airplanes could land upon an airport in such areas even if built on this A-7 soil. And yet, this same soil in New York State, subject to freezing and thawing, would

TABLE

SOIL CLASSIFICATION FOR

1		2	3	4		5
Major Divisions		Soil Groups and Typical Names	Suggested Group Symbols	General Identification		Observations and Tests Relating to Material in Place
				Dry strength	Other pertinent exams.	
Coarse-Grained Soils	Gravel and gravelly soils	Well-graded gravel and gravel-sand mixtures, little or no fines	GW	None	Gradation, grain shape	Dry unit weight or void ratio, degree of compaction, cementation, durability of grains, stratification and drainage characteristics, ground-water conditions, traffic tests, large scale load tests or California bearing tests
		Well-graded gravel-sand-clay mixtures, excellent binder	GC	Medium to high	Gradation, grain shape, binder exam. wet and dry	
		Poorly graded gravel and gravel-sand mixtures, little or no fines	GP	None	Gradation, grain shape	
		Gravel with fines, very silty gravel, clayey gravel, poorly graded gravel-sand-clay mixtures	GF	Very slight to high	Gradation, grain shape, binder exam. wet and dry	
	Sands and sandy soils	Well-graded sands and gravelly sands, little or no fines	SW	None	Gradation, grain shape	
		Well-graded sand-clay mixtures, excellent binder	SC	Medium to high	Gradation, grain shape, binder exam. wet and dry	
		Poorly graded sands, little or no fines	SP	None	Gradation, grain shape	
		Sand with fines, very silty sands, clayey sands, poorly graded sand-clay mixtures	SF	Very slight to high	Gradation, grain shape, binder exam. wet and dry	
Fine-Grained Soils Containing Little or No Coarse-Grained Material	Fine-grained soils having low to medium compressibility	Silts (inorganic) and very fine sands, Mo, rock flour, silty or clayey fine sands with slight plasticity	ML	Very slight to medium	Examination wet (shaking test and plasticity)	Dry unit weight, water content and void ratio, consistency—undisturbed and remolded, stratification, root holes, fissures, etc., drainage and ground-water conditions, traffic tests, large scale load tests, California bearing tests or compression tests
		Clays (inorganic) of low to medium plasticity, sandy clays, silty clays, lean clays	CL	Medium to high	Examination in plastic range	
		Organic silts and organic silt-clays of low plasticity	OL	Slight to medium	Examination in plastic range, odor	
	Fine-grained soils having high compressibility	Micaceous or diatomaceous fine sandy and silty soils, elastic silts	MH	Very slight to medium	Examination wet (shaking test and plasticity)	
		Clays (inorganic) of high plasticity, fat clays	CH	High	Examination in plastic range	
		Organic clays of medium to high plasticity	OH	High	Examination in plastic range, odor	
Fibrous organic soils with very high compressibility		Peat and other highly organic swamp soils	Pt	Readily identified		Consistency, texture and

LEGEND FOR GROUP SYMBOLS: G, gravel; S, sand; M, Mo, very fine sand, silt, rock flour; C, clay; Pt, peat; F, fines, matl. <0.1 mm.; O, organic; W, well graded; P, poorly graded; L, low to medium compressibility; H, high compressibility.

XI

AIRPORT PROJECTS *

6	7	8	9	10	11	12	13	14	15
Principal Classification Tests (on Disturbed Samples)	Value as Foundation When Not Subject to Frost Action	Value as base Directly Under Wearing Surface	Potential Frost Action	Shrinkage, Expansion, Elasticity	Drainage Characteristics	Compaction Characteristics and Equipment	Solids at Opt Compaction (lb./cu. ft.) and Void Ratio e	Cal. Bearing Ratio for Compacted and Soaked Specimen	Comparable Groups in Public Roads Classification
Mechanical analysis	Excellent	Good to excellent	None to very slight	Almost none	Excellent	Excellent, tractor	>125 $e<0.35$	>50	A-3
Mechanical analysis, liquid and plastic limits on binder	Excellent	Fair to excellent	Medium	Very slight	Practically impervious	Excellent, tamping roller	>130 $e<0.30$	>40	A-1
Mechanical analysis	Good to excellent	Poor to good	None to very slight	Almost none	Excellent	Good, tractor	>115 $e<0.45$	25–60	A-3
Mechanical analysis, liquid and plastic limits on binder if applicable	Good to excellent	Poor to good	Slight to medium	Almost none to slight	Fair to practically impervious	Good, close control essential, rubber-tired roller, tractor	>120 $e<0.40$	>20	A-2
Mechanical analysis	Good to excellent	Poor to good	None to very slight	Almost none	Excellent	Excellent, tractor	>120 $e<0.40$	20–60	A-3
Mechanical analysis, liquid and plastic limits on binder	Good to excellent	Poor to good	Medium	Very slight	Practically impervious	Excellent, tamping roller	>125 $e<0.35$	20–60	A-1
Mechanical analysis	Fair to good	Not suitable	None to very slight	Almost none	Excellent	Good, tractor	>100 $e<0.70$	10–30	A-3
Mechanical analysis, liquid and plastic limits on binder if applicable	Fair to good	Not suitable	Slight to high	Almost none to medium	Fair to practically impervious	Good, close control essential, rubber-tired roller	>105 $e<0.60$	8–30	A-2
Mechanical analysis, liquid and plastic limits if applicable	Fair to poor	Not suitable	Medium to very high	Slight to medium	Fair to poor	Good to poor, close control essential, rubber-tired roller	>100 $e<0.70$	6–25	A-4 A-6 A-7
Liquid and plastic limits	Fair to poor	Not suitable	Medium to high	Medium	Practically impervious	Fair to good, tamping roller	>100 $e<0.70$	4–15	A-4 A-6 A-7
Liquid and plastic limits from natural condition and after oven drying	Poor	Not suitable	Medium to high	Medium to high	Poor	Fair to poor, tamping roller	>90 $e<0.90$	3–8	A-4 A-7
Mechanical analysis, liquid and plastic limits if applicable	Poor	Not suitable	Medium to very high	High	Fair to poor	Poor to very poor	<100 $e>0.70$	<7	A-5
Liquid and plastic limits	Poor to very poor	Not suitable	Medium	High	Practically impervious	Fair to poor tamping roller	>90 $e<0.90$	<6	A-6 A-7
Liquid and plastic limits from natural condition and after oven drying	Very poor	Not suitable	Medium	High	Practically impervious	Poor to very poor	<100 $e>0.70$	<4	A-7 A-8
natural water content	Extremely poor	Not suitable	Slight	Very high	Fair to poor	Compaction not practical			A-8

NOTE: In Column 7, values are for subgrade and base courses, except for base courses directly under wearing surface
NOTE: Values in Columns 7 and 8 are for guidance only. Design should be based on test results in accordance with text.
NOTE: Unit weights in Column 13 apply only to soils with specific gravities ranging between 2.65 and 2.75.

* Courtesy of Army Engineering Manual, Chapter XX, Part II, Exhibit 1.

not give sufficient support to the heaviest duty highway pavement to withstand the repeated loading of a medium bomber during spring thaws.

93. The Soils Engineer. The engineer, therefore, who would build airports must be not only an accomplished soils engineer from a laboratory standpoint, but he must also have a complete understanding of what the actual measurements mean under different conditions of use. He should be able to make soil analyses not only with the full equipment which he finds in the laboratory in America, but also with the very limited equipment which he may find at his disposal under combat conditions in foreign lands.

94. Elimination of Binder Material. It should be noted also that too much reliance cannot be placed upon the mere classification of soils into the respective A-1 to A-8 groups, because they do not necessarily mean what some people interpret them to mean. Soils are too frequently thought of in respect to their behavior as surface courses. The term soils means, of course, the gravels, sands, sand clays, and similar soil combinations which have been used in secondary road construction. The fact that they may work satisfactorily as a surfacing material does not mean that they always will be satisfactory for bases for, as a matter of fact, in any areas subject to frost binder material (passing 200 mesh) should be largely eliminated. The less 200 mesh material there is retained in base courses of granular character, consistent with ability to place and compact, the better. With cohesive soils a different criterion may apply. These undergo relatively little volume change when frozen and thus ice lenses do not occur to as great an extent as in sand gravels containing too much fine material. With cohesive soils the rate of formation of ice lenses is dependent on the porosity and rate of capillary flow of moisture to feed the formation of the ice lenses.

95. Bearing Power. The bearing power of the subgrade must be known before the engineer is able to make a scientific design for a pavement. By pavement is meant the composite structure of wearing surface, base, and subbase. It is necessary to know what the bearing power will be under the worst condition of use, and here again the engineer must have knowledge as to the probable changes in behavior throughout the year. The mere fact that a soil has bearing power of say 50 pounds per square inch at optimum moisture content is no criterion of what it will have when saturated. On the other hand, it may never become saturated (as in barren areas) and therefore to design an airport pavement on a constant rule of testing a soaked soil is just as much an error as to design it upon dry conditions alone. Proper evaluation of local conditions must always be made. The making of bearing tests for airports is not always a simple matter. The location may be difficult to reach and to apply a load of 60,000 to 75,000 pounds requires much manipulation of equipment. However, the costs involved in a large airport are so great as to justify fully very complete preliminary studies in regard to subgrade behavior.

One of the needs is a method for more rapid making of tests, so that at least five to ten a day can be made. It is apparent that tests upon the subgrade may be made more rapidly than upon the pavement, because of the lesser load intensities. As a rule, once the subgrade bearing power has been determined over the area involved, trial pavement sections are needed in only two or three thicknesses to determine the minimum and maximum requirements. The improvement in bearing power resulting from the surcharge effect of the overlying pavement is often very large. This is especially true of clean sands and gravels which only require a thin pavement to permit them to give rocklike behavior.

96. Apparatus for Tests. Several kinds of apparatus have been used for field testing. In one type, two axles of the kind used to transport telephone poles are fastened together with 12-inch I-beams, the flanges touching, to form a trailer. The span is about 20 feet.

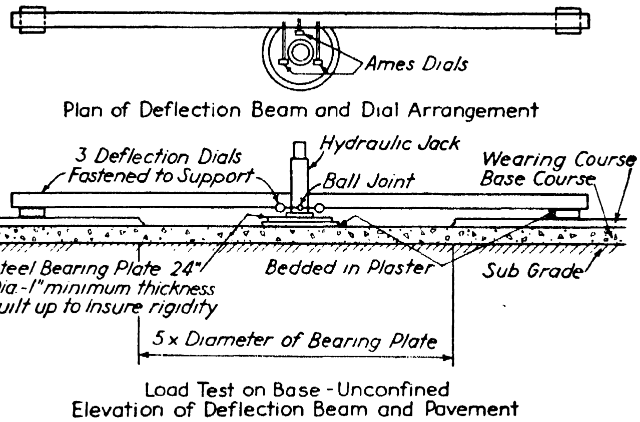
A box, made of 12-inch channels or I-beams, is welded transversely under the center of the beams to provide the bearing against which the hydraulic jack is operated. Similar boxes are provided at either end for resting on four other jacks in order to take the load off the tires, which otherwise would burst under the heavy loads. The hydraulic jack has a pressure gage attached and, together with suitable other gages, it provides for obtaining the necessary data for drawing a load-deflection curve.

Another method which shows much promise requires the use of a cupola-like truss whose four corners are attached to soil anchors. The same type of hydraulic jack is employed; it is placed against the underside of the truss, and it is estimated that the soil anchors properly placed will develop as much as 60,000 pounds of resistance each before pulling out.

A great advantage of this method would be the lighter weight and consequently easier manipulation, and very probably a greater number of tests per day, especially if some device were invented for rapidly placing the soil anchors.

With the steel beam trailer, about three tests per day are possible, with loads up to 60,000 pounds. Where building is actually under way, the large construction equipment can be used to jack against and will be heavy enough to determine qualities of base and subgrade as a rule. The Omaha, Nebraska, Airport was built under such control. The bearing plates upon which the hydraulic jack rests are made to correspond to the tire contact area and usually equal 18, 24, and 30 inches in diameter respectively. Only the plate corresponding to largest contact tire area needs to be used. It is to be remembered that the loading test on a steel plate is more severe than loading on a rubber tire. It is essential to have a competent man in charge.

Perhaps a few comments in respect to technique of loading tests will be of interest. Strange as it may seem, there have been numerous tests run where the soil was not in any way prepared or compacted prior to making measurements. Obviously, where a load-deflection curve is to be obtained, the degree of consolidation prior to the test is of the greatest importance. What is desired is the yield point of both the subgrade soil and the composite pavement

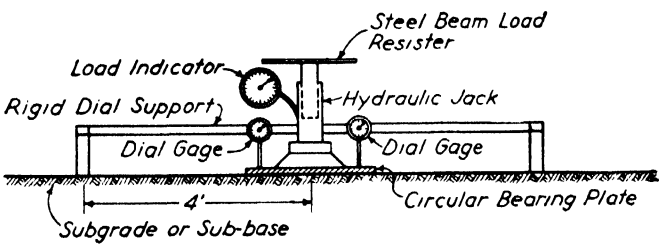


Courtesy Asphalt Institute

FIG. 69. An alternate method of assembling equipment.

which is under test. Accordingly, either the soil should be compacted on the test area first, to represent ultimate conditions after construction, or else the full test load should be applied once to simulate the consolidation likely to occur. The compressed area may then be evened up with fine sand or with plaster of Paris to insure a proper bearing for the steel bearing plate.

The actual test load then is applied in increments of approximately 5 pounds per square inch until a deflection of

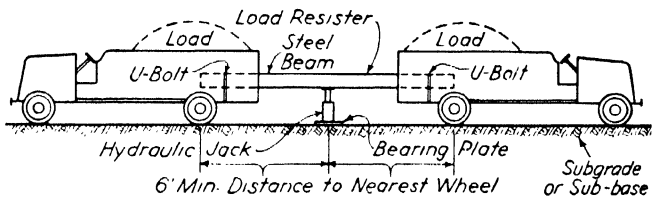


Courtesy Asphalt Institute

FIG. 70. Hydraulic jack, gages, bearing plate for recording resistance of subgrade.

0.5 inch has been obtained. Such a deflection is greater than some subgrades and pavements can stand without plastic flow. A clean dry sand or gravel can stand almost any deflection without lateral flow. On the other hand, a wet silty clay may start to flow at 0.2 inch deflection or even less. Hot-mix asphaltic concrete, for example, fails in tension on the lower side at approximately 0.5 inch deflection. Other softer asphalt types, however, will not stand this amount. The controlling factor, then, is the deflection which the weakest element in the composite structure can stand for the unit load which it will be called upon to sustain in its position in the structure. A soil which will deflect 0.5 inch under a 30-pound loading at the surface, however, may deflect but 0.2 inch at a foot below the surface under the same load, because of the beneficial confining effect of the overlying layers. Therefore it is important to know the bearing value of soils both confined and unconfined, and hence the importance of actual field bearing tests.

The Army engineers are attempting to work out a design procedure and have tentatively adopted the so-called California method of appraising the bearing power of soils. This method, which was devised by Mr. O. J. Porter, shows



Courtesy Asphalt Institute

FIG. 71. General arrangement of testing equipment.

excellent correlation of test procedure with California service behavior for highway loadings. However, the attempt to project highway experience to produce curves for airport loadings obviously has its limitations, and whether they are too large or too little will be found out only through actual service behavior. The Army engineers recognize this and are endeavoring to correlate the California laboratory tests with actual field bearing tests as rapidly as possible. Much progress has been made already, and the Corps of Engineers is to be congratulated and commended for the enterprise shown in this work.

At a recent meeting of the Committee on Flexible Design of the Highway Board, a thorough review was made

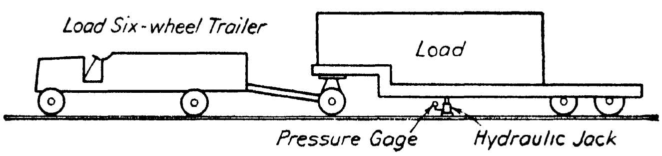


FIG. 72. One method of attaining required load.

of all the methods of preliminary design for both airports and highways, and it was concluded that for wheel loads up to 15,000 pounds our present highway experience is probably adequate, but that for loadings beyond this amount field bearing tests always should be made and that, no matter what the cost, the resultant economies and assurance of adequacy would fully warrant such initial study. Some of the various formulas which have been

developed for highway design are applicable for predetermining the thickness of the trial pavement which is to be constructed for the field test. In general, pavement thickness varies as the square root of the load, plus or minus some factor. Usually two or three trial sections will be sufficient.

97. Structural Requirements of Road Beds. When the foregoing tests are made many soils will not have the desired bearing power and it will be necessary to improve them by some method or treatment before a pavement surface can be laid. It may be said that the structural requirements of a good road bed are as follows.

1. Compressive strength in excess of that of the maximum wheel load to be used.
2. Effective pore space so limited in volume that it cannot receive sufficient water to produce plasticity.
3. Water content so restricted in quantity as to cause the road to remain an elastic solid.
4. Surface protection against the destructive forces of traffic, weather, and erosion.

To satisfy the foregoing requirements both designer and builder must possess adequate knowledge of the structural requirements of road beds and must be familiar with the mechanical properties of the available materials.

98. Air Content. During the period of construction the engineer is concerned with the amount of air in earth materials. In case of freezing, the air is either compressed or removed, and room is thereby furnished for the expansion of freezing water within the pores without disrupting the solid mass. A limited air content in the fine pores of a road bed is an element of safety. A compacted embankment will contain about 2 per cent of air voids.

99. Water Content. When freezing water is rigidly confined, its force is enormously destructive. The only known method of avoiding this destruction to foundations or surfaces is that of restricting the quantity of water. The forces exerted by water are both gravitational and molecular. The molecular forces include adhesion, cohesion, and chemical affinity. Water is essential at all times as a constituent but excessive use of it is to be avoided. "Spare the water and increase the weight of the roller" is generally a safe maxim when compacting earth.

100. Aggregate. A solid aggregate is one which offers resistance to any force that tends to change either its form or its size.

Hardness is that property of a material that causes it to resist deformation or abrasion by external forces.

Brittleness is characteristic of materials having little or no plasticity. Dry clays are characteristically brittle. Dry particles are easily sheared; they form dust and therefore produce traffic hazards.

Elasticity is that property of materials which causes them to recover or tend to recover their original form or size upon removal of the deforming force. When the pressure exceeds that of the elastic limit of a given material, plastic flow begins. One of the essentials of any traveled way, whether it is a path, a road, or an airport runway, is that of elasticity.

The traveled way must be able to resist wheel loads adequately and also to recover after their passage.

Inasmuch as bone-dry clays are characteristically brittle and wet clays plastic, there must be a state of moisture at which a given clay may be made to attain its maximum elasticity.

Plasticity produces a permanent deformation under an applied load. It has been defined as that property by means of which a material may be deformed or changed in shape and retain that shape when the deforming force is removed.

Yield point is the point at which wet materials change from the elastic to the plastic state and is expressed as a percentage of the water content.

101. Granular Materials. Sand and gravel deposits are largely granular and are characterized by their limited compressibility and the ease with which they receive and lose water.

Loose material, regardless of other attendant structural properties, should have no place in a foundation or wearing surface. Pore space is always large in loose materials and, when the individual opening is so small that the force of surface tension is greater than that of gravity, water is removed only through evaporation.

Dry earth should be used only for blending or conditioning wet earth so as to obtain little variation of moisture throughout the structure.

Wet clay, whether in a plastic or fluid condition, is generally in a state of expansion; it has very little supporting capacity and therefore should not be used.

102. Stabilization. There are two main points to keep in mind when a design for a stabilized surface is contemplated.

1. It is necessary to prevent the detrimental volume change of soil which, if allowed, would distort a pavement.

2. The soil must be stabilized to support a flexible type of pavement, as this type depends almost entirely upon the base for its stability.

Although there are many variables which tend to complicate the treatment and manipulation of soils an engineer should have a thorough understanding of certain fundamental facts. These facts are grouped and stated briefly in the following four statements.

1. The engineer should have an understanding of the characteristics of soil materials.

2. The engineer should have a knowledge of soil compaction.

3. The engineer should have some intimation of natural colloidal cements and the effect of admixtures on soil materials.

4. The engineer should have an accurate conception of capillary phenomena in soils.

For the present discussion we shall think of soil materials as falling in one of two groups.

1. The gravels, sand, and silt which are the products of nature. They are formed by freezing and thawing, wind, and erosion by water. This group in general has particles larger than 0.005 mm. Crushed stone and slag might also be included in this group.
2. The clays, colloids, etc., are included in this second group.

Each of these groups may be broken down into two classes:

(*a*) Those which are at all soluble in water.
(*b*) Those which are not soluble in water.

Limestone, slags, and the like are soluble enough when wet to have thin films of gelatinous cement on their surfaces which bind the particles upon drying.

Certain granites are insoluble.

Soluble aggregates, when properly graded and treated chemically, become highly stable. Calcium chloride, sodium chloride, and other chemical solutions increase the solubility of the soluble rocks and therefore release more gelatinous cement to bind the particles into stable masses.

Insoluble rock becomes unstable unless improved with small amounts of soluble rock powder.

Clays and colloids result from the chemical action of weathering and plant growth and decay, and consequently might differ considerably in chemical composition from their parent rock. Clays and colloids differ in another and very important respect from silt, sand, and gravel. Sand has a very definite grain size whereas clay may not. The size recorded by sieve analysis for clay may not be the size of a particle but rather the size of aggregations of clay and colloidal particles. Therefore, the clay fraction must be considered as a material with exceedingly variable properties, as compared with the gravel, sand, and silt fractions which have definite grain sizes and consequently relatively constant properties.

The whole theory of stabilization depends upon the ability to change the character of clays by treatment.

103. Compaction. It would be better to specify the density of a built-up fill rather than the thickness as the density determines the supporting power of the soil.

Since soils must be taken out of cuts and placed in fills, compaction by rollers and sheep's-foot rollers is necessary to obtain good consolidation. The key to success in this process is to employ some method to determine the proper amount of moisture to use to obtain the desired density.

104. Proctor Density Test. *Optimum moisture content* is that moisture content in a soil at which maximum density of the soil is obtained. There are several methods used to determine the density of a soil sample and since density is one of the desired conditions we shall describe one of these methods which has been used extensively.

The so-called Proctor density test may be made in the field or in the laboratory and does not require a large amount of equipment.

105. The Proctor Equipment. 1. A metal cylinder 4½ inches high with an inside diameter of 4 inches.

2. A flat metal base to support the cylinder.

3. A filling collar 2 inches high.

4. A 5½-pound cylindrical rammer with an end area of 3 square inches.

5. A set of plasticity needles; sizes 0.01, 0.1, 1.0 inch.

6. A penetration resistance device to determine the pressure required to force a needle of known end area into a compacted soil sample at a given rate.

The procedure for making the test is outlined as follows.

Procedure

1. A 6-pound sample, air dried to slightly damp condition, is taken from a portion of the material passing the No. 4 sieve.

2. The sample is next thoroughly mixed, moistened slightly, and packed into the cylinder in three layers. Each layer is tamped with 25 blows from the tamper which is dropped from a height of 1 ft.

3. The soil is then struck off to the level of the cylinder and the weight is determined.

4. A reading is then taken with the penetration resistance device, the density being measured by the force required for the needle to penetrate at a rate of ½ in. per sec. The pressure is recorded in pounds per square inch. Since the needles come in different sizes, a computation is necessary to reduce the pressure to pounds per square inch.

5. A small sample of the soil is next oven dried and weighed to get the moisture content.

6. The soil is then removed from the cylinder and broken up until it will pass the No. 4 sieve. Water is added in a little larger proportion and the entire procedure is repeated. This procedure is continued until enough data are collected to plot a *moisture-density curve.*

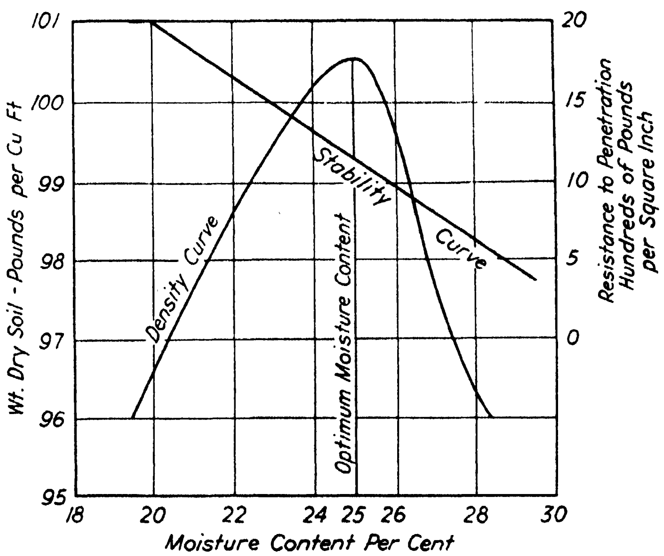


FIG. 73. Moisture-density curve.

The effect of admixtures on the density of stabilized soil may be illustrated by means of these tests. They are based on the fact that for each soil there is but one moisture content, called the optimum moisture content, at which the maximum density is produced by a specific amount of compacting. For each density, each soil has a particular stability as indicated by the force required to penetrate the soil with a footing of known area at a given speed. When the soil is compacted to maximum density at optimum moisture content, the adsorptive attraction between water and soil particles is probably such that the tendency for moisture to enter the soil and expand or soften the soil mass is largely eliminated.

From the foregoing discussion it will be seen that we know how to produce a given compaction but we have little information as to how long the condition will remain. Some soils swell and swell means a drop in stability. To complete subgrade stabilization, we must not only compact the soils properly but also introduce admixtures which will provide against swelling of the soil afterwards.

106. Admixtures. Admixtures may be classified as (1) those to improve the mechanical grading; (2) those to furnish binding properties; (3) those to improve the quality of the soil binders; (4) those to reduce the capillary properties.

The kinds of admixtures and the means by which they effect soil stabilization are summarized below (from Public Roads, Vol. 17, No. 3).

1. Mineral aggregate and soil constituents of the character and size required to make graded mixtures stable.
2. Moisture-retentive chemicals, such as calcium chloride and common salt, to provide soil binders with enough moisture to facilitate the compaction of graded mixtures by traffic.
3. Solutions of electrolytes such as calcium chloride, common salt, sodium hyposulphite, etc., to reduce the thickness of adhesive water films on soil particles and thus provide stabilized mixtures with greater density.
4. Primes and fillers such as soaps, stone dust, and slag to increase the adhesion between mineral constituents and the chemical and bituminous admixtures and thus assist in retaining the benefits the admixtures were expected to furnish.
5. Neutralizers such as limestone dust, slag, hydrated lime, etc., that serve to alkalize acid soils and thus prevent the loss of stabilizing chemicals caused by detrimental base exchange.[1]
6. Water-insoluble binders, such as portland cement and bituminous materials, to furnish films more substantial than those of moisture alone and to destroy permanently the colloidal properties responsible for detrimental volume change in soil mixtures.

[1] Base exchange occurs when adsorption involves reactions that are essentially chemical or ionic in character. (Adsorption is the phenomenon that causes all solids to tend to adsorb or condense on their surfaces any gases or vapors with which they are in contact.)

107. Mechanical Stabilization. A high density is a virtual necessity where granular materials are to be used as surface courses. Graded-soil-type base courses under flexible pavements must possess the required stability to support wheel loads. If the stabilized material is used in the surface course it must in addition withstand the abrasion of the traffic. It should also shed water and yet retain dampness, by capillary action, to replace the moisture lost through evaporation.

Standard specifications are set up for both surface courses and base courses. These specifications are given in terms of the amount of the material passing a series of sieve sizes.

108. Gradation Charts. These are made up from the standard specifications and are usually made similar to Figs. 74 through 77.

109. Proposed A.A.S.H.O. Standard Specifications for Materials for Stabilized Base Course Specification. Those presented in the Association's specification M56-38 follow.

A. Material Covered

1. This specification covers the quality and size of sand-clay mixtures; gravel, stone or slag screenings, or sand, crusher run coarse aggregate consisting of gravel, crushed stone or slag combined with soil mortar or any combination of these materials for use in the construction of a stabilized base course. The requirements are intended to cover only materials having normal or average specific gravity, absorption and gradation characteristics. Where materials such as caliche, gypsum, limerock and water soluble salts are to be used appropriate limits suitable to their use must be specified.

B. Types

2. The following types of base course stabilized mixtures are specified. The Engineer shall designate the type or types desired:

Type A—Sand-clay mortar.
Type B—Coarse graded aggregate.
Type C—Gravel, stone or slag screenings or sand.

C. General Requirements

3. The type or types designated shall conform to the following requirements:

Type A. The materials for this type shall be composed of natural or artificial mixtures of clay or soil binder and gravel, sand or other aggregate proportioned to meet the requirements hereinafter specified. The aggregate retained on the No. 4 sieve shall be composed of hard, durable particles and shall be free from injurious or deleterious substances.

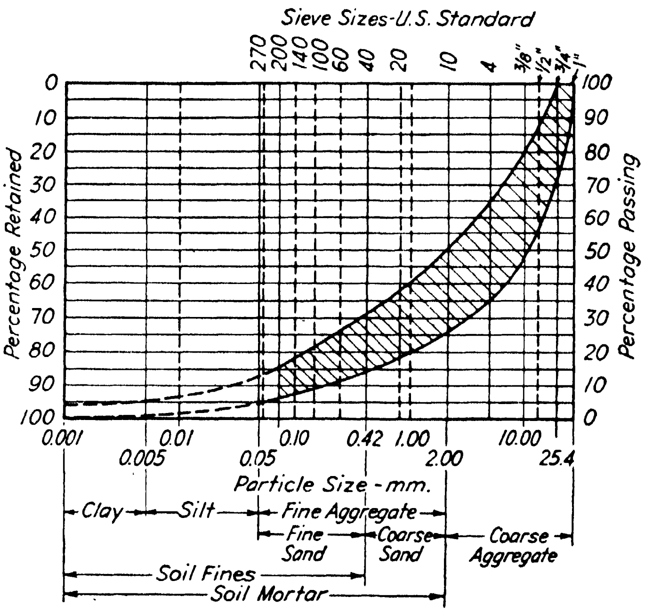


FIG. 74. Stabilization chart for 1 inch maximum size aggregate for base course.

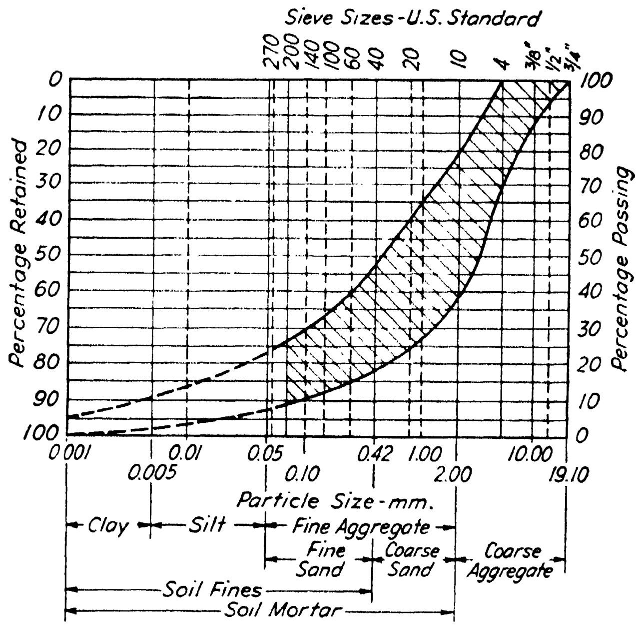


FIG. 75. Stabilization chart for fine aggregate mixtures.

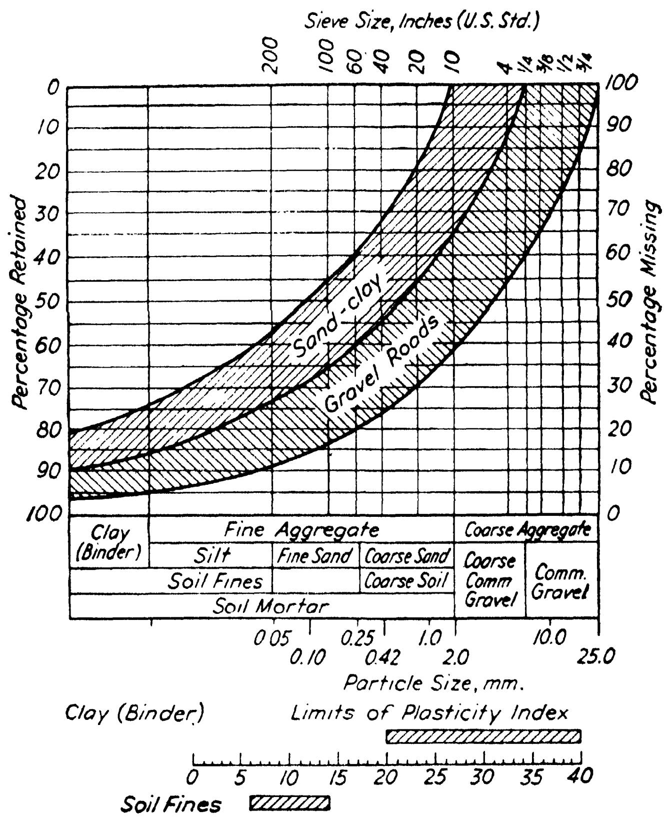


FIG. 76. Gradation charts for sand-clay and gravel surfaces.

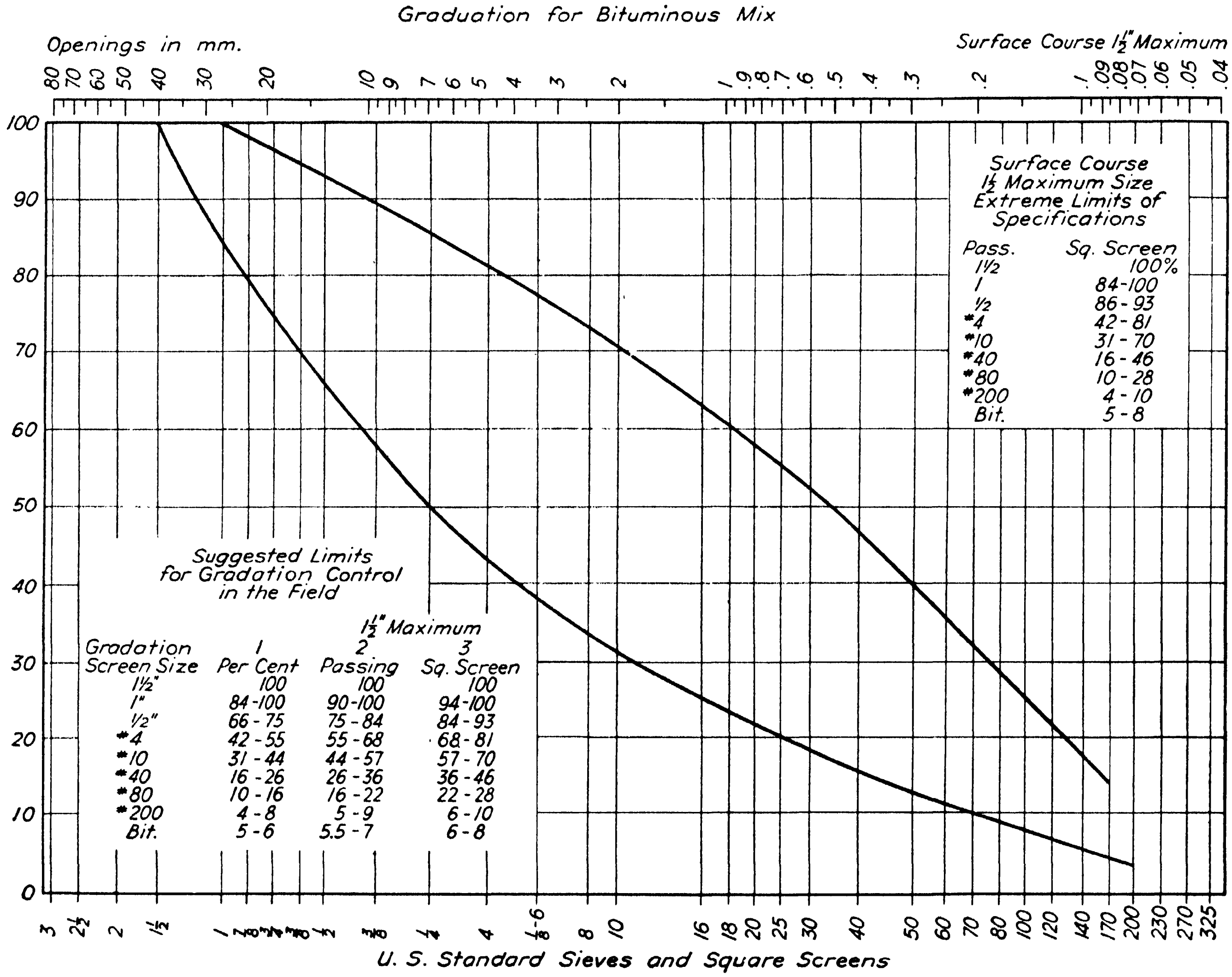


FIG. 77.

Type B. The material for this type shall consist of natural or artificial mixtures of gravel, stone or slag and soil mortar so proportioned as to meet all the requirements hereinafter specified.

The coarse aggregate shall consist of clean, hard, durable particles of crushed or uncrushed gravel, stone or slag free from soft, thin elongated or laminated pieces and vegetable or other deleterious substances. It shall be hard and durable enough to resist weathering, traffic abrasion and crushing. Shales and similar materials that break up and weather rapidly when alternately frozen and thawed or wetted and dried, should not be used.

The soil mortar shall be that portion passing the No. 10 sieve and shall be composed of soil binder and granular material such as stone or slag screenings or sand.

Type C. The materials for this type shall be composed of gravel, stone or slag screenings or sand or mixtures thereof proportioned to meet the requirements hereinafter specified.

The material shall be composed of hard, durable particles, free from injurious or deleterious substances, uniformly graded from coarse to fine.

4. The type or types designated shall conform to the following requirements:

Type A

Passing	Percentage by Weight
1-inch sieve	100
No. 10 sieve	65–100

The material passing the No. 10 sieve shall meet the following requirements:

Passing	Percentage by Weight
No. 10 sieve	100
No. 20 sieve	55–90
No. 40 sieve	35–70
No. 200 sieve	8–25

The fraction passing the No. 200 sieve shall not be greater than ½ the fraction passing the No. 40 sieve. The fraction passing the

No. 40 sieve shall have a liquid limit not greater than 25 and a plasticity index not greater than 6.

Type B

Passing	B-1 1-In. Max. Size	B-2 2-In. Max. Size
	Percentage by Weight	
2-inch sieve		100
1½-inch sieve		70–100
1-inch sieve	100	55–85
¾-inch sieve	70–100	50–80
⅜-inch sieve	50–80	40–70
No. 4 sieve	35–65	30–60
No. 10 sieve	25–50	20–50
No. 40 sieve	15–30	10–30
No. 200 sieve	5–15	5–15

The fraction passing the No. 200 mesh sieve shall be less than ½ of the fraction passing the No. 40 sieve. The fraction passing the No. 40 sieve shall have a liquid limit not greater than 25 and a plasticity index not greater than 6.

Type C

Passing	Percentage by Weight
¾-inch sieve	100
No. 4 sieve	70–100
No. 10 sieve	35–80
No. 40 sieve	25–50
No. 200 sieve	8–25

The fraction passing the No. 200 sieve shall be less than ½ of the fraction passing the No. 40 sieve. The fraction passing the No. 40 sieve shall have a liquid limit not greater than 25 and a plasticity index not greater than 3.

D. Moisture Content

5. The materials A, B, and C herein specified shall contain sufficient moisture to insure maximum compaction.

E. Admixtures

6. Chemicals or other admixtures shall meet all the requirements of the current A.A.S.H.O. specifications. When the chemical to be used is not covered by A.A.S.H.O. specifications, a good commercial grade meeting the approval of the engineer shall be used.

F. Methods of Testing

7. Sampling and testing shall be in accordance with the following standard methods of the A.A.S.H.O.

(*a*) Sampling	T 2-35
(*b*) Size	T 27-38
(*c*) Liquid limit	T 89-38
(*d*) Plasticity index	T 91-38

110. Chemical Stabilization. It has been mentioned before that moisture is desirable for good stabilization. Under many temperature and climatic conditions evaporation may take place to such an extent that no moisture remains in the aggregate. Calcium chloride has been used with temporary success and, since moisture promotes cohesion, of first importance is its ability to retard evaporation, thereby keeping the material moist. Calcium chloride also attracts additional moisture; that is, it not only retards the rate of evaporation from the surface during the heat of the day, but also, because of its deliquescent character, actually recaptures or replaces lost moisture at night, or under other favorable humidity conditions. The ability of calcium chloride to attract moisture varies according to the climatic conditions under which it is used. It is soluble in water and moves along with soil moisture either in the capillary or free water state.

111. Some Common Admixtures. Asphalt, tars, and portland cement are used under various conditions for stabilizing aggregate. They are the most expensive from the standpoint of first cost but other methods may prove unsatisfactory. Sufficient admixtures must be used to insure the required bearing ratio. These admixtures give best results if the soil is thoroughly pulverized and thoroughly mixed with the admixtures.

112. Factors in Soil Stabilization. Some of the methods of soil stabilization which are quite suitable under one condition of soil or one condition of climate may be actually detrimental under wet weather construction conditions. For example, an airport was constructed on a swamp area with 36 inches of bank-run gravel placed as a sub-base. This gravel was quite free from 200-mesh material and at first was rather loose and slow to compact. However, it could stand unlimited rainfall without breaking up and gradually was made smooth and ready for the base course. The base course consisted of a commercial type of stabilization course 8 inches thick; the same gravel was used but a certain amount of fines was added to increase density. This base was placed rapidly and appeared to be stable as it was supporting heavy trucks without rutting. The surface course consisted of 1½ inches of an open-graded stone plant-mix. After the surface was placed, its sieve-like character permitted water to be trapped and the surface behaved like a blotter instead of a seal. The course had not been completely stabilized and could still take up water, so in a very short time it had become softened to such a degree that even a light truck rutted the surface. Upon digging through this base course, however, it was found that the sub-base was completely unaffected because it contained no fines, and was so hard that it was difficult to remove with a pick.

Several things were wrong with such a design for the particular location. First, the mixed-in-place form of stabilization should not have been specified for the wet weather conditions which were to be expected. In all probability, a base course of the gravel alone, with just a little more attention to gradation, would have been all

that was required. Second, the open-graded top was not suitable for an airport surface because it is porous and, because there is no traffic to close the voids as there would be on a highway, the surface did not properly protect the base.

In this particular case, the following procedure rectified the situation. The porous top was filled with sand; a heavy seal coat was then applied so that no more water could get into the base. The sub-base gradually took up the surplus water from the base course until finally the whole structure came into equilibrium and adequate support was obtained. If this sub-base, however, had contained a large percentage of fines which would have made it capillary in behavior, the readjustment of moisture content could not have occurred, and the only remedy then would have been to scarify completely and reconstruct both base and top.

Frequently, one will see a macadam base being placed during wet weather upon a subgrade which is cut by ruts to a depth of several inches. This means that the bottom 2 or 3 inches of stone are lost so far as their contribution to bearing power is concerned. Under such conditions, an insulation course consisting of fine granular material such as stone screening or sand should be placed first. This will overcome the soft condition of the subgrade and provide a firm working table upon which to place the crushed stone course and thereby make the layer completely effective.

From the foregoing, it is clear that in the construction of an airport the following considerations are of primary importance. First consideration should be given to the drainage; this will insure a uniform subgrade support. Second, the base course must be of a material unaffected by weather changes and in itself largely capable of carrying the loads which will come upon it. This is sound procedure at any time because, as a rule, it will be the most economical one. Under wartime conditions, however, it becomes mandatory because the shortage of various kinds of manufactured products in many areas makes it necessary to conserve transportation in every possible way. Regardless of pavement type, whether flexible or rigid, the improvement of the subgrade and placement of base courses will lead to a minimum section for the surface course.

It is often possible to construct these subgrades and bases in such a manner that a simple asphalt surface treatment will take care of the situation as far as load support alone is concerned, and for a period of time such a surface treatment may take care of emergency operations. On heavy duty fields, however, the abrasion and torsional stresses must be taken into account, and if a surface treatment only is to be employed, it is necessary to have the upper part of the base course constructed of crushed aggregate so that high interlocking can be secured.

Chapter VIII

Flexible-Type Pavements

113. Turf Surfaces. There are sections of the country where soil and climatic conditions are such that a surface of graded turf will furnish an adequate material for runways. This type of surface is only good for relatively light traffic and a good growth of grass is necessary to keep down the dust and prevent erosion of the surface by rain and wind. Many foreign airports were built of turf but it can be said that it is not a suitable surface for heavily traveled runways.

Practically every airport requires special attention to determine the grasses that are best suited for the particular climate and soil conditions. The young grass must be protected until it has become thoroughly rooted; this means that no traffic can use certain parts of the runways while the seed is getting its start.

Under heavy traffic, turf is likely to become rutted, especially if exposed to alternate freezing and thawing. During the wet seasons, the maintenance of airplanes becomes more expensive, landings more difficult, and accidents more numerous.

As a matter of economy, a turf surface may be put down but most runways will eventually be paved with an artificial surface.

114. Hard Surfacing for Runways. Runway paving is quite generally used in airport construction and it may be defined as the process of preparing a firm, stable, even, all-year, all-weather surface, free from dust or aggregate which may be blown or picked up by airplane engine propellers. It must also be designed to support the static and dynamic loads of the planes which are to use the airport.

The selection of a type of surface will depend upon many factors, a few of which are given in the following list:

1. Subgrade conditions (very important).
2. Load requirements.
3. Permanence of surface desired.
4. Materials and equipment available.
5. Present and future traffic.
6. Temperature and precipitation.
7. Available funds for the work.
8. Engineering practices in the community.

The types of surfaces are similar to those used in highway practice and as yet there has been no specific type developed especially for airport runways. Much heavier loads can be expected on runways and this may change the design but, in general, only the thickness will be affected. It is far better to build up a relatively high bearing subsoil in the base course and keep the pavement as thin as possible.

Pavements may be divided into the two general groups:

1. Rigid.
2. Non-rigid or flexible.

The rigid pavements are portland cement concrete, soil cement, certain asphaltic concrete, and water-bound macadam.

The non-rigid or flexible pavements include practically all the bituminous types, such as those constructed with cutbacks or emulsions, those mixed in place, those using plant mix, penetration, and bituminous stabilization of native soils.

Longitudinal and transverse grades for airport runways are specified by the C.A.A. and are tabulated in Table II, page 6. The Army air force has made slight modifications in the recommended standards, as will be noted in Table XII.

Runway grades should permit an unobstructed view from any point 10 feet above the surface of a runway to any other point 10 feet above the runway.

TABLE XII

RUNWAY AND SHOULDER GRADES

Distance between center line parallel runways	500 ft. minimum
Distance between center line of runway and airport building	750 ft. minimum
Distance between edge of aprons or taxiways and building or obstructions	75 ft. minimum
Distance between center line of runway to parking apron, railroad, or road	500 ft. minimum
Landing strip and runway grades—transverse	1½% maximum
Landing strip and runway grades—longitudinal	1% maximum (except where excessive grading is required. Never exceed 1½%.)
Longitudinal grade intersections joined by vertical curves	300 ft. minimum
Longitudinal tangent intervals between vertical curves	1000 ft. minimum
Grade changes in runways	½ of 1% maximum

115. Typical Cross Sections. The cross sections adopted for a runway may vary but they will have the general form shown in Figs. 78 to 86.

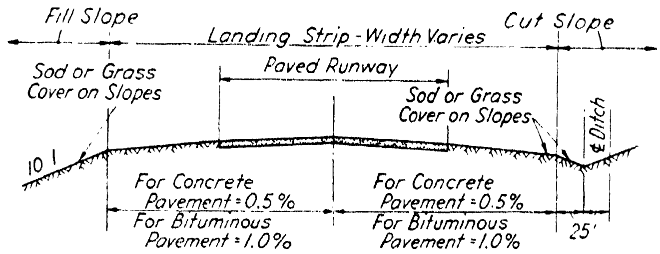


FIG. 80. Crowned runway.

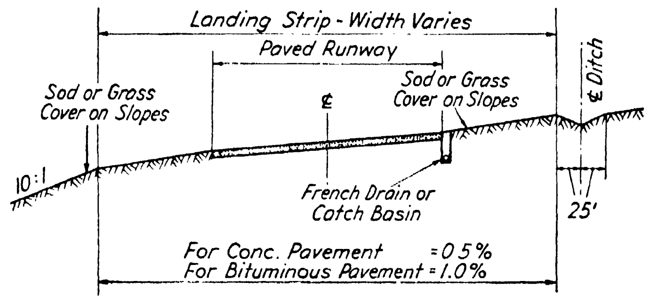


FIG. 81 Straight line crown runway.

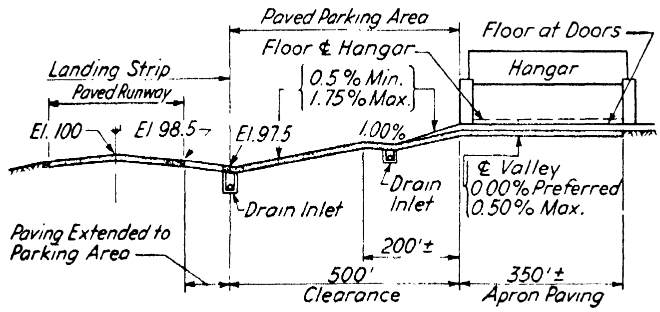


FIG. 82. Maximum grades to elevated hangars.

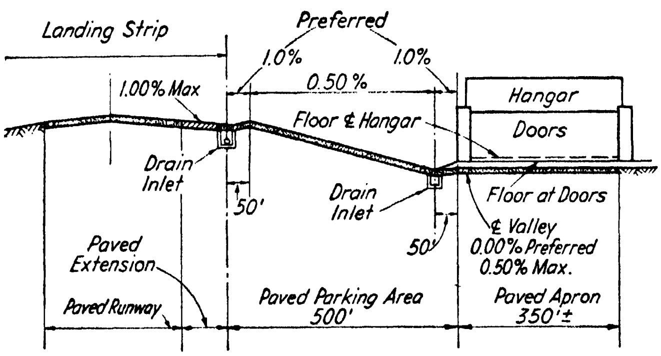
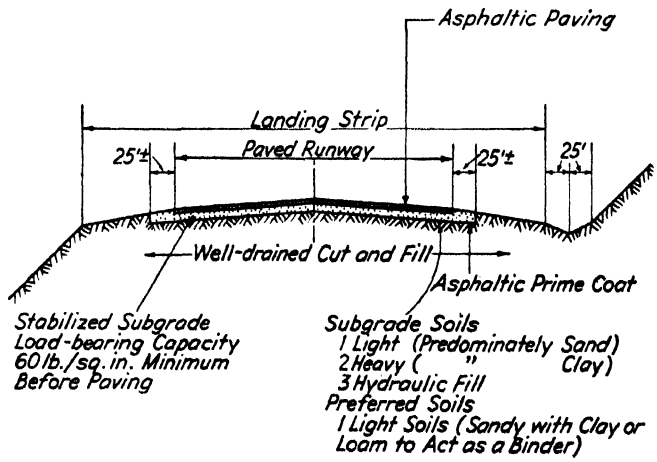


FIG. 78. Maximum grades to depressed hangars.



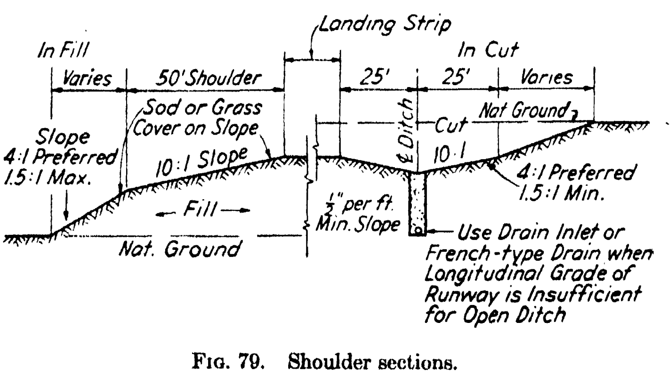


FIG. 79. Shoulder sections.

FIG. 83. Typical cross section.

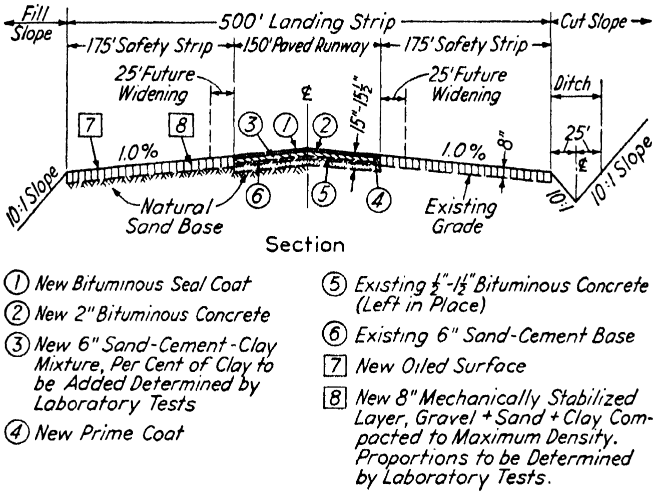


FIG. 84. Improvement of existing pavement.

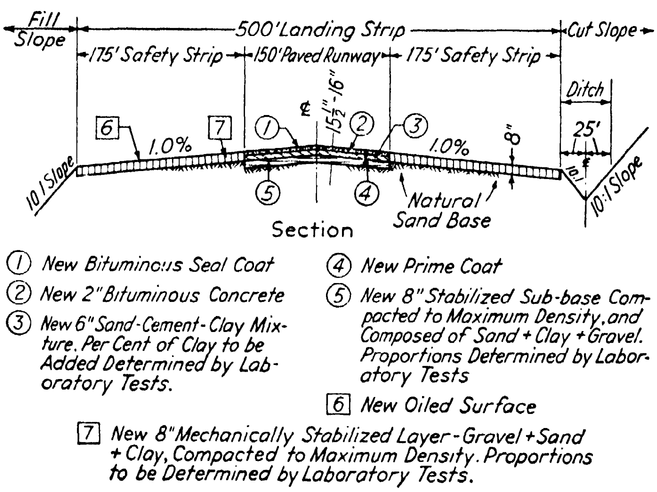


FIG. 85. New pavement for runways, taxiways, and aprons.

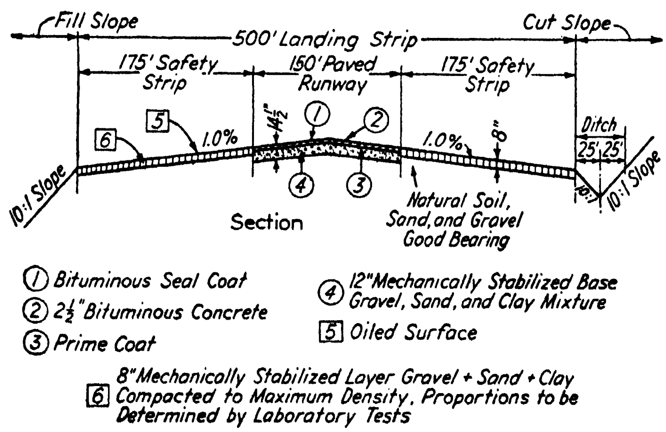


FIG. 86. Pavement with improved safety strips.

116. Thickness of Pavement. In general it may be stated that a design for the cross section of a runway consists of a compacted, stabilized subgrade, a base course, and a wearing surface. The cross section must be designed to support the loads expected on the finished surface.

Several different theories have been developed and it is interesting to compare these theories with those used on highway work. A transition from the early theories has taken place mainly because of the increased loads and the use of large pneumatic tires.

The following theories apply to a flexible pavement where the loads are transmitted to the subgrade. A rigid pavement will support some of the load and a different theory of design is used for this type. The flexible pavements include various forms of macadam and gravel surfaces, bituminous macadam, and bituminous concrete as well as most stabilized bases. Some engineers feel that bituminous concrete and similar pavements develop a considerable amount of resistance to bending under moving loads but this is not considered in the development of the following formulas.

117. Massachusetts Highway Formula. Probably the earliest theory for the design of a flexible pavement was

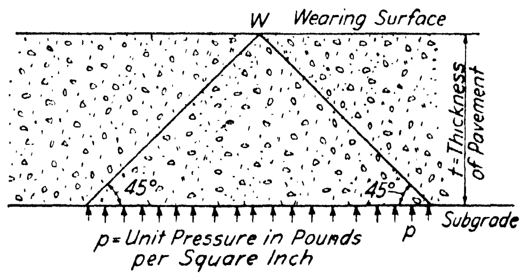


FIG. 87. Distribution of load to subgrade.

that known as the Massachusetts Highway Formula. In the days when this formula for thickness was developed most of the traffic was horse drawn and the loads were transmitted to the road surface at a point or very small area. This load was considered to be transmitted to the subgrade along the lines of a square pyramid and spread 45° with the horizontal as shown in Fig. 87. The conditions of Fig. 88 then existed and the following development of an equation for the thickness resulted.

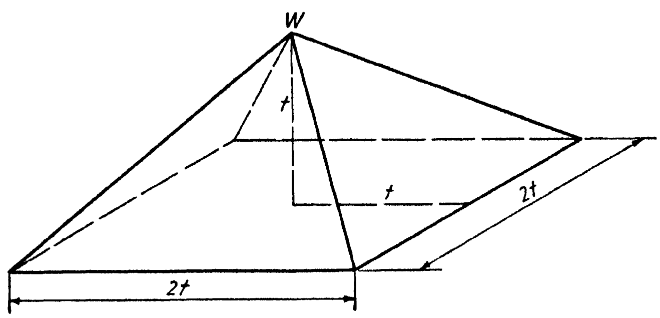


FIG. 88. Load distributed over an area.

Since the area of the base of this pyramid is $4t^2$ it becomes a footing which rests on the subgrade and will support a load equal to the bearing power in pounds per square inch of the subgrade multiplied by the area in square inches or $4t^2p$. The load is W pounds. Therefore we may equate

$$W = 4t^2p$$

or

$$t = \sqrt{\frac{W}{4p}}$$

which is the Massachusetts Highway Formula.

The use of this formula may be illustrated by the following problem:

Problem

Find the thickness of a pavement for a weight of 20 tons in a truck with $^6\!/_{10}$ of the load supported on the rear axle. The bearing power of the subsoil is 50 lb./sq. in.

$$t = \sqrt{\frac{W}{4p}} = \sqrt{\frac{0.5 \times 0.6 \times 40{,}000}{4 \times 50}} = \sqrt{60} = 7.7 \text{ or } 8 \text{ in. approx.}$$

118. Harger and Bonney Formula. The Massachusetts Highway Formula gives thick pavements; when this was realized modification was made in the Harger and Bonney Formula to adapt a formula to the wide steel tires necessarily used to support heavier loads.

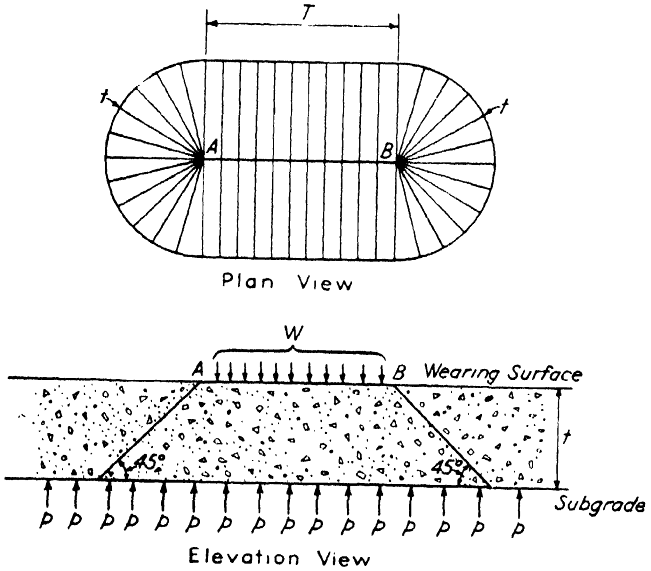


FIG. 89. Harger and Bonney method.

In the development of this formula T was taken as the width of a steel tire in inches and the load was applied along a line AB.

The area in contact with the subgrade is composed of a rectangle and two half circles attached at each end of this rectangle.

The area of the rectangle $= 2tT$.

The area of the two half circles $= \pi t^2 = 3t^2$, approximately.

The supporting surface multiplied by the bearing power p is then:

$$p(2tT + 3t^2)$$

This must support the load W.

$$\therefore W = p(2tT + 3t^2)$$

The value of t, the thickness of the pavement in inches, is desired.

$$W = 2Tpt + 3pt^2$$

$$3pt^2 + 2Tpt = W$$

$$t^2 + \frac{2Tt}{3} + \left(\frac{T}{3}\right)^2 = \frac{W}{3p} + \left(\frac{T}{3}\right)^2$$

$$t + \frac{T}{3} = \pm\sqrt{\frac{W}{3p} + \frac{T^2}{9}}$$

$$t = \sqrt{\frac{W}{3p} + \frac{T^2}{9}} - \frac{T}{3}$$

119. Down's Formula. Down's Formula was developed by using a theory similar to the Massachusetts Formula except that a cone was used rather than the square pyramid. Figure 90 illustrates the following derivation.

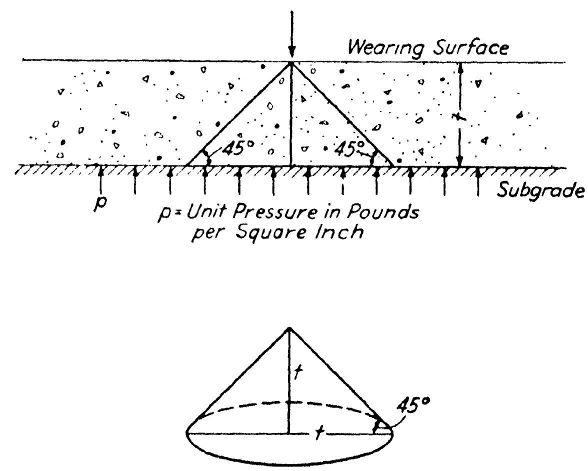


FIG. 90. Down's method.

The area in contact with the subgrade is πt^2 and this multiplied by the bearing power p gives the supporting power for the thickness t.

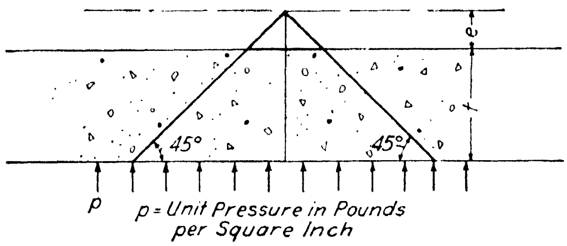
$$W = \pi t^2 p$$

or

$$t = \sqrt{\frac{W}{\pi p}} = 0.564\sqrt{\frac{W}{p}}$$

120. Asphalt Institute Formula. In all the formulas thus far discussed, the load has been applied at a point or

along a line. These are not the true conditions when pneumatic tires are used. Modern trucks and airplanes are all equipped with tires which spread the load over a circle or an ellipse. A circle was used in the formula devised by the Asphalt Institute. (See Fig. 91.)



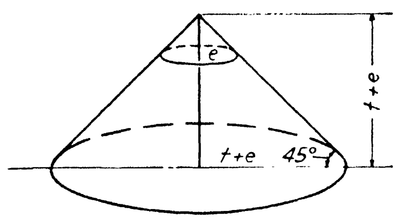


FIG. 91. Asphalt Institute method.

The radius of the circle of contact of the tire with the wearing surface is e and, therefore, the radius of the circle of contact of the base of the cone is $(t + e)$.

Applying the same principles of the preceding derivations,

$$W = \pi(t + e)^2 p$$

$$(t + e)^2 = \frac{W}{\pi p}$$

$$t + e = \sqrt{\frac{W}{\pi p}} = 0.564\sqrt{\frac{W}{p}}$$

$$t = 0.564\sqrt{\frac{W}{p}} - e$$

121. Goldbeck Formula. Larger pneumatic tires make a contact area on a pavement which is more nearly elliptical than circular. This fact has been demonstrated by actual imprints made on pavement surfaces and therefore it would seem that there is a need for the development of a formula for thickness of pavement which will take this fact into consideration.

The Goldbeck Formula does consider the elliptical area of contact and, although this may not be the final solution, it is given here to complete the cycle of thought which started with the Massachusetts Highway Formula in the early days of flexible pavements.

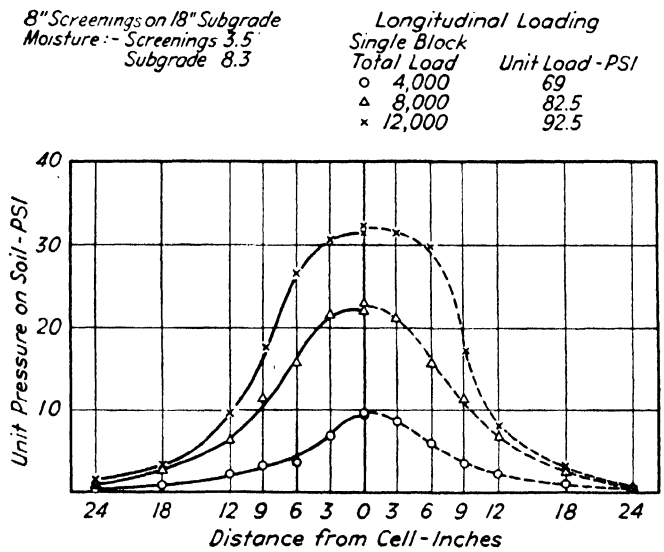
122. Subgrade Pressures. Since most of the load on a flexible pavement is transmitted to the subgrade, some information about the distribution of the pressure on the subgrade is necessary. Pressure cells have been used to determine this distribution and the results are indicated in Fig. 92.

These curves show typical pressure conditions of the distribution of the pressure on subgrades for 4000-, 8000-, and 12,000-pound loads transmitted through 8 inches of very stable limestone screenings. The maximum pressures shown in Fig. 92 will be greater or less, depending in part on such conditions as:

1. The thickness of the non-rigid surface.
2. The stability of rigidity of that surface.
3. The magnitude of the wheel load.
4. The area of the imprint of the tire.
5. The load-supporting value of the subgrade.

The maximum pressure intensity on the subgrade produced by a wheel load should not exceed the bearing capacity of the soil.

Bearing Capacity. A subgrade soil acts partly as an elastic and partly as a plastic material. Elasticity means that a soil will recover after a load is released. Plastic means that a soil will retain a permanent deformation after a load is released. These conditions may be demon-

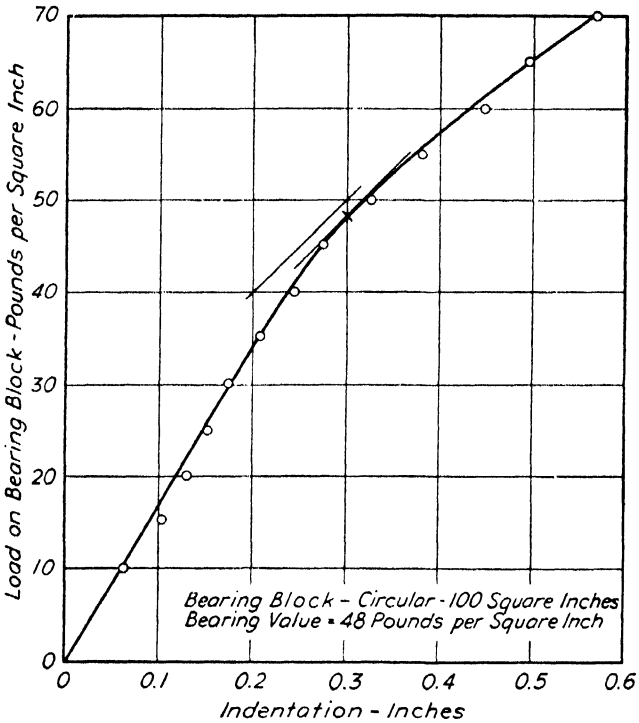


Courtesy National Crushed Stone Association

FIG. 92. Soil pressures.

strated by tests using bearing blocks and a compression testing machine.

Figure 93, by Goldbeck, illustrates the results of bearing value test on a subgrade with a circular bearing block with an area of 100 square inches.



Courtesy National Crushed Stone Association

FIG 93. Bearing value test on subgrade.

The indentation of the bearing block depends on

1. The magnitude of the load.
2. The duration of application.
3. The size of the bearing block.

The sharp break in the curve at about 48 pounds per square inch indicates the yield point. The block, under these tests, is forced into the material being tested and therefore produces an indentation. A small indentation is not serious because the material, as a result of the elasticity of the subgrade material, may recover after the load is released. Some doubt exists as to just how much indentation is allowable but experience indicates that about ½ inch is permissible.

From a study of the curve in Fig. 93 it will be seen that the curve breaks at a point somewhere between 40 and 50 pounds per square inch. The point where the ratio of pressure increase to indentation increase is equal to 100 will give a closer value for design purposes. This point may be also described as the point where a 45° line becomes tangent to the curve of Fig. 93 and is about 48 pounds per square inch.

123. Thickness Formula (Single-Tire Load). The actual pressure in a subsoil was indicated in Fig. 92 and is also shown in Fig. 94.

The following equation then results from an analysis similar to that used in deriving the Harger and Bonney Formula. From Fig. 94,

$$W = \pi(L_2 + t)(L_1 + t)p$$

$$(L_2 + t)(L_1 + t) = \frac{W}{\pi p}$$

$$L_2L_1 + L_2t + L_1t + t^2 = \frac{W}{\pi p}$$

$$t^2 + (L_1 + L_2)t + \left[\frac{L_1 + L_2}{2}\right]^2 = \frac{W}{\pi p} - L_1L_2 + \left[\frac{L_1 + L_2}{2}\right]^2$$

$$\therefore t = \sqrt{\frac{W}{\pi p} - L_1L_2 + \left[\frac{L_1 + L_2}{2}\right]^2} - \frac{L_1 + L_2}{2}$$

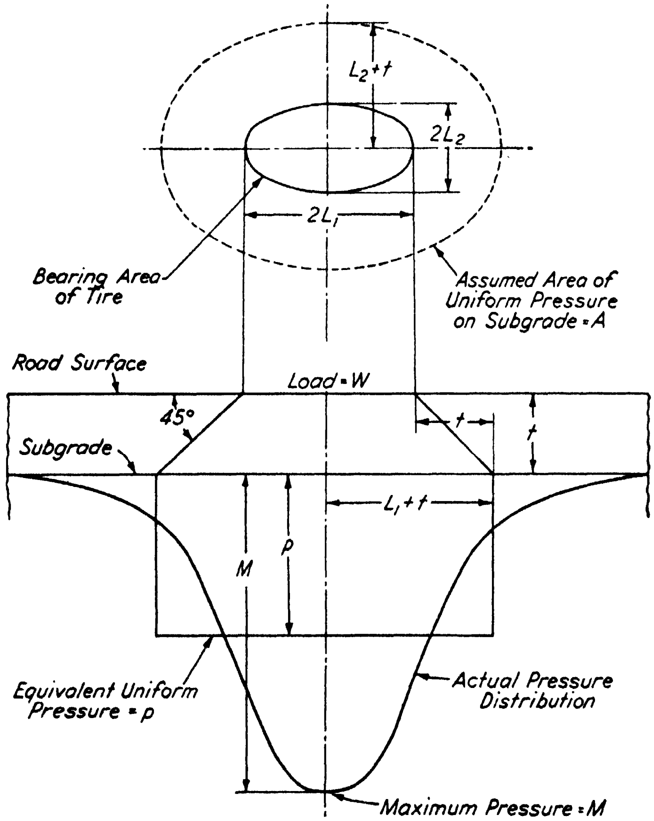
Since

$$K = \frac{M}{p}$$

$$p = \frac{M}{K}$$

then

$$t = \sqrt{\frac{KW}{\pi M} - L_1L_2 + \left[\frac{L_1 + L_2}{2}\right]^2} - \frac{L_1 + L_2}{2}$$



Courtesy National Crushed Stone Association

t = thickness of non-rigid pavement
W = the maximum wheel load on a single pneumatic tire
M = maximum pressure intensity on the subgrade
p = the calculated equivalent uniform pressure on subgrade
K = M/p
L_1 = ½ the major axis of ellipse of tire contact area
L_2 = ½ the minor axis of ellipse of tire contact area

FIG. 94. Case of single-tire load.

The value of K in the above formula is determined by experiment and additional experiments will undoubtedly give a more precise value for each kind of surfacing material. If the surfacing material is rigid the value of K will be smaller than for surfaces which become less rigid with the addition of water.

This method of design is less theoretical than some others but it is practical and can be applied easily. There is some doubt as to the justification of the use of the highly theoretical methods because of the large number of variables and the great variety of soils and combinations of materials of construction. At least, this method is a working basis for design which may be refined and corrected as more information on the behavior of soils is obtained.

Values of K as suggested by the National Crushed Stone Association are shown in Table XIII.

TABLE XIII

RATIO k OF MAXIMUM SUBGRADE PRESSURE M TO CALCULATED UNIFORM PRESSURE p

$$k = \frac{M}{p}$$

Test No.	Subgrade		Surface			Single-Tire Loads, lb.			Dual-Tire Load, lb.
	Per cent moisture	Bearing value lb./sq. in.	Type	Per cent moisture	Thickness in.	4000	8000	12,000	8000
						Values for k			
			Stable Surfaces						
2	9.4	56	Screenings	3.4	4	1.9	2.0	...	2.8
11	9.1	70	Screenings—emulsion	2.8	4	1.5	1.6	1.7	...
12A	8.8	60	Water-bound macadam	2.0	4	1.4	1.8	1.8	2.5
1	9.2	50	Screenings	3.8	6	1.6	1.6	1.9	...
12B	8.2	..	Water-bound macadam	1.2	6	1.0	...	...	1.0
14	10.2	48	Sand-clay-gravel	3.3	6	2.5	2.5	2.7	2.4
13A	9.6	30	Water-bound macadam	2.0	10	2.3	3.1	2.8	2.6
15	10.3	50	Sand-clay-gravel	3.7	10	1.6	1.9	2.1	1.8
16	....	40	Hot. bit. conc.	...	4.6	2.0	2.7	2.4	2.6
13B	10.3	44	Water-bound macadam	2.7	10	1.8	2.0	2.0	2.3
10	10.8	15	Screenings	2.9	4	1.4	1.6	...	...
10A	11.5	10	Screenings	2.1	4	2.1	...	...	...
7	11.0	30	Screenings	2.8	6	...	1.3	1.4	1.4
12	10.4	29* (−)	Water-bound macadam	1.1	6	...	2.0	2.2	...
8	10.3	49	Screenings	2.8	8	...	1.2	1.5	1.3
13	10.3	25	Water-bound macadam	1.5	10	1.6	2.1	2.2	2.2
11A	9.9	42	Screenings—emulsion	3.8	4	1.9	2.0	2.2	...
6	8.3	100	Screenings	3.5	8	1.3	2.0	2.0	1.6
					Ave.	1.7	2.0	2.1	2.0
			Unstable Surfaces †						
3	10.5	44	Screenings	6.2	4	4.3	...	...	4.4
5	10.4	55	Screenings	7.0	8	4.4	5.3	...	3.2
9	10.0	40	Screenings	6.0	6	4.5	4.1	...	3.8
15B	9.5	50	Sand-clay-gravel	4.5	10	2.7	3.3	3.6	2.6
					Ave.	4.0	4.2	3.6	3.5

* At ½-in. indentation, the ratio of pressure increase to indentation increase is only 55 instead of desired minimum of 100. At no value of indentation was subgrade sufficiently resistant.

† These surfaces were purposely made unstable by the use of excessive water in their preparation.

Courtesy of National Crushed Stone Association.

The average value K is 2 and this value is sufficiently accurate for general design purposes. For airplane tires it is best to use $K = 2$ and calculate the values of L_1 and L_2 from the known wheel-load and tire pressure.

The width of the elliptical area of contact of an airplane tire may be taken as the nominal width of tire ($2L_2$).

The area of the ellipse of contact $= \pi L_1 L_2$. Then $\pi L_1 L_2 \times$ inflation pressure = actual tire load.

Example

Let it be supposed that an airplane runway is to be designed for a gross weight of 45,000 lb.

Tire size = 19.00 × 23

Inflation pressure = 63 lb./sq. in.

Maximum subgrade resistance = 40 lb./sq. in. = M

$K = 2$

Then

$$2L_2 = 19.00$$

$$L_2 = 9.5 \text{ in}$$

$$\pi \times 9.5 \times L_1 \times 63 = 22{,}500$$

$$L_1 = \frac{22{,}500}{\pi \times 9.5 \times 63} = 12 \text{ in.}$$

$$t = \sqrt{\frac{KW}{\pi M} - L_1L_2 + \left[\frac{L_1 + L_2}{2}\right]^2} - \frac{L_1 + L_2}{2}$$

$$t = \sqrt{\frac{2.0 \times 22{,}500}{\pi \times 40} - 12 \times 9.5 + \left[\frac{12 + 9.5}{2}\right]^2} - \frac{12 + 9.5}{2}$$

$t = 8$ in. approx.

TABLE XIV

AIRPLANE TIRE PRESSURES (HIGH PRESSURE)

Size	Number of Plies	Air Pressure
26 × 6	6	75
26 × 6	8	85
30 × 7	8	85
32 × 8	8	90
34 × 9	9	50
34 × 9	10	80
38 × 10	10	80

TABLE XV

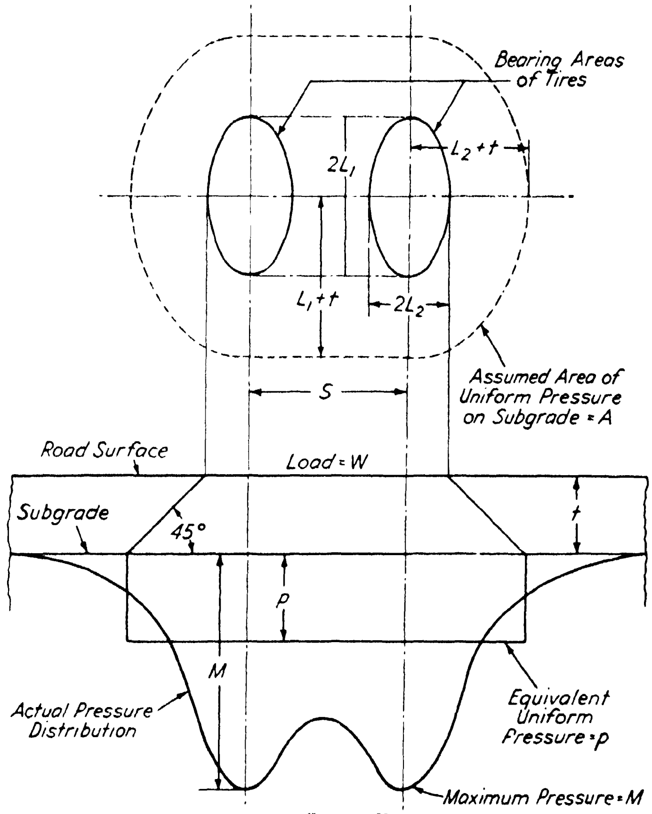
AIRPLANE TIRE PRESSURE (LOW PRESSURE)

Size	Number of Plies	Air Pressure
15.5–16	12	62
16.0–16	10	43
17.0–16	10	48
18.0–16	10	48
17.0–20	12	70
19.0–23	14	55
19.0–23	16	63

Tables XIV and XV give a few sizes of high and low pressure airplane tires and the air pressure in pounds per square inch used in the different sizes.

Pressures that exceed 100 pounds per square inch are seldom, if ever, used because the relative pressure is increased tremendously at high altitudes where the outside atmospheric pressure is much less than at ground level where the tire is inflated. The tires for high altitude flying must be checked very carefully as blow-outs may occur from over-inflation.

124. Dual-Tire Loads. The Goldbeck theory that has been discussed with its relation to single tires may be



Courtesy National Crushed Stone Association

FIG. 95. Case of dual-tire load.

modified to apply to dual tires. Most of the larger airplanes will require four-wheel supports and the wheels will be grouped in pairs as shown in Fig. 95.

The wheel load in this case is less severe than when carried on a single tire because:

1. The load is spread over a wider area on the subgrade.
2. The tire pressure is less in the smaller tires used together.

The following derivation refers to Fig. 95.

t = thickness of pavement as before.
W = maximum wheel load on a pair of dual wheels.
M = maximum pressure intensity on subgrade.
p = calculated equivalent uniform pressure on subgrade.
$K = M/p$.

$L_1 = \frac{1}{2}$ the major axis of ellipse of tire contact area.
$L_2 = \frac{1}{2}$ the minor axis of ellipse of tire contact area.
S = center-to-center spacing of dual tires.
A = area of equivalent subgrade pressure.

$$A = 2S(L_1 + t) + \pi(L_1 + t)(L_2 + t)$$

$$W = Ap = \frac{AM}{K} = [2S(L_1 + t) + \pi(L_1 + t)(L_2 + t)]\frac{M}{K}$$

Solving for t

$$\frac{KW}{M} = [2SL_1 + 2St + \pi(L_1L_2 + L_2t + L_1t + t^2)]$$

$$= [2SL_1 + \pi L_1L_2 + (2S + L_2 + L_1)t + t^2]$$

$$\frac{KW}{M} - 2SL_1 - \pi L_1L_2 = t^2 + (2S + L_1 + L_2)t$$

Complete the square, transpose, and solve for t:

$$t^2 + (2S + L_1 + L_2)t + \left[\frac{2S + L_1 + L_2}{2}\right]^2 = \frac{KW}{M} - 2SL_1 - \pi L_1L_2 + \left[\frac{2S + L_1 + L_2}{2}\right]^2$$

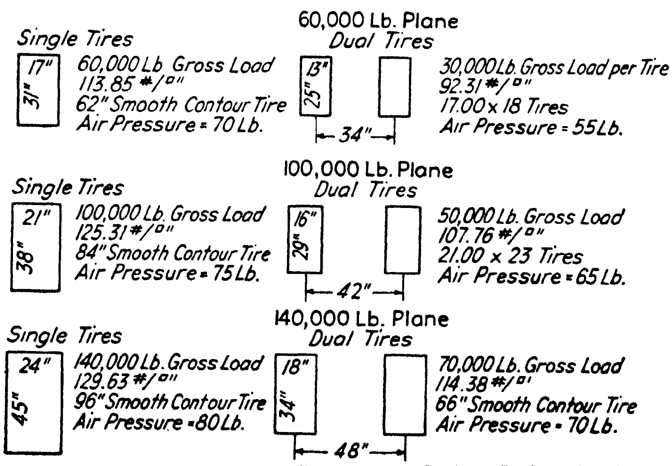
Take the square root of both sides:

$$t + \frac{2S + L_1 + L_2}{2} = \sqrt{\frac{KW}{M} - 2SL_1 - \pi L_1L_2 + \left[\frac{2S + L_1 + L_2}{2}\right]^2}$$

then

$$t = \sqrt{\frac{KW}{M} - 2SL_1 - \pi L_1L_2 + \left[\frac{2S + L_1 + L_2}{2}\right]^2} - \frac{2S + L_1 + L_2}{2}$$

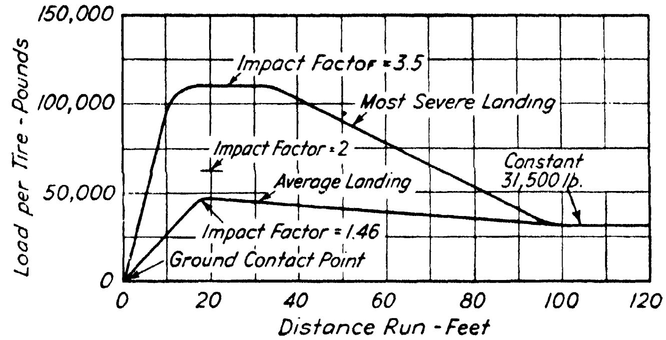
125. The Spacing of Dual Tires. In Fig. 96 are illustrated some typical loadings on single and dual tires with the spacing for dual-tire combinations.



Courtesy Armco Drainage Products Association

FIG. 96. Typical tire loading.

126. Loads and Impact. The landing load per tire on a runway may be illustrated as in Fig. 97. From Fig. 97 it might be concluded that impact is very large for a landing



Courtesy Armco Drainage Products Association

FIG. 97. Landing loads from a 63,000-pound plane, single tires. (From a study by E. A. Raymond, Douglas Aircraft Co.)

plane but analysis reveals that it may not be as much as first supposed. Owing to a flattening of the pneumatic tire the area of contact of a tire on the runway is increased many times as the plane wheel makes contact with the runway surface. The area under normal load with the plane at rest may be about 450 square inches; this area may be increased to 1000 square inches upon the first contact while landing. The load, in pounds per square inch, is therefore not so great as might at first be supposed. There are some differences of opinion among engineers as to whether or not impact should be considered under these conditions. Impact or, better, a safety factor should be considered in the design of all hard standing and warm-up areas. The vibration of planes during warm-up has a decided detrimental effect on a pavement. Some engineers use a safety factor of 50 per cent, whereas others recommend a safety factor 100 per cent of the load applied on a wheel.

127. Asphalt Pavements. From the foregoing it can be seen that various theories have been developed for the thickness design of a flexible pavement, with reference to subgrade support and traffic loads. Investigations are being carried on to study the mechanism of failure due to inadequate subgrade support when the flexible pavement is subjected to concentrated loading.

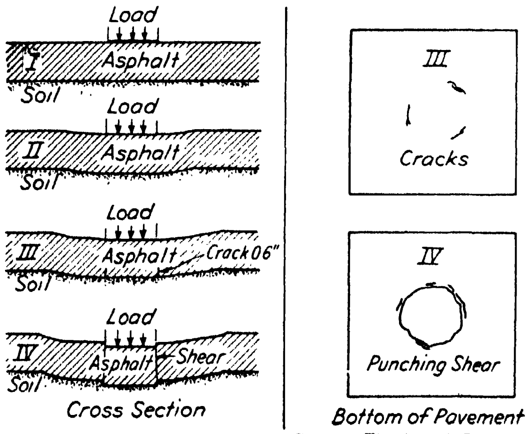
Actual tests and observation of the behavior of asphalt pavements resting directly upon soil masses have been made by the Asphalt Institute. The preliminary investigation was confined to hot-mix types of pavements such as an asphaltic concrete and sheet asphalt.

128. Failure of Asphalt Pavement when Inadequately Supported by the Subgrade. The following statements are quoted from Research Series No. 7, a publication of The Asphalt Institute.

Any properly designed asphalt pavement of whatever thickness will successfully carry the heaviest traffic load if it rests upon an

unyielding base. This can be readily demonstrated by subjecting a thin pavement section to concentrated loading when placed directly on the base of a compression testing machine.

If the same section is, however, placed upon a mass of plastic soil and again tested it may fail under a relatively small concentrated load. In such case when the load is gradually increased over a bearing area, the pavement adjacent to the testing head develops a cup-shaped depression which increases in depth until failure in punching shear occurs. Numerous tests have demonstrated that irrespective of variations in area of testing head or in thickness of pavement, both asphaltic concrete and sheet asphalt withstand a deflection of 0.5 inch directly under the loaded area with no evidence of failure but that at 0.6 inch deflection, punching shear either occurs or is imminent as evidenced by the development of cracks on the bottom side. [Figure 98 illustrates such typical behavior.]



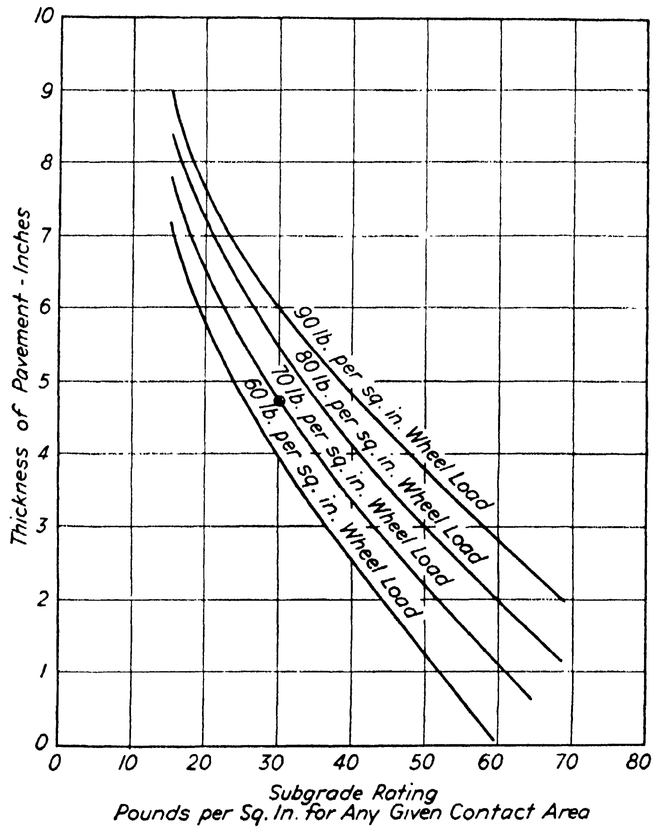
Courtesy The Asphalt Institute

FIG. 98. Progressive failure under concentrated loading.

The development of cracks under concentrated loading represents failure of the pavement, although it is true that for a time at least the pavement may actually be able to support heavier loads. The approximate maximum load which it will carry without cracking is that which produces a deflection of ½ inch and the load causing this deflection has therefore been taken as the primary basis of evaluating the pavement with reference to its subgrade support. If this basis is accepted, resistance of the soil mass to the same critical deflection may be taken as its supporting value with reference to the pavement.

In many instances it will be found that the load required to produce a deflection of 0.5 inch in the soil is far less than the maximum load-supporting capacity of the soil. The pavement may therefore fail before the subgrade is stressed to its ultimate load-supporting capacity. The fact that failure of the pavement is primarily due to a critical deflection of the subgrade rather than to its maximum capacity for carrying loads greatly simplifies the problem of determining thickness of pavement required to carry traffic loads with reference to subgrade support.

129. Thickness Design Curves. As a result of the investigations of the Asphalt Institute, the curves in Fig. 99 have been suggested as a guide for determining the thickness of an asphaltic concrete pavement.



Courtesy The Asphalt Institute

FIG. 99. Thickness design curves.

130. Non-Rigid or Flexible Pavements. In general, it has been found that 2 inches of asphalt pavement is about the required minimum when anything thicker than a surface treatment is to be paved; moreover, for a heavy duty field, this surface should be a hot-mix type, preferably of a coarse graded asphaltic concrete. On the other hand, there is no point in making such an asphalt surface of greater thickness than 5 inches because it is usually more economical to secure any needed additional bearing power through the use of cheaper aggregates in the foundation. Likewise, with the employment of crushed aggregate, there is no particular point in utilizing thicknesses greater than 12 inches for even the heaviest loadings, because at depths of more than 1 foot sufficiently high bearing value for the lower horizons can be obtained through placement of layers of such materials as sand, sandy gravel, slag or cinders, and selected soils.

We therefore have the design of hot asphalt pavements made from the surface down, with the range between 2 and 5 inches in accordance with the availability of foundation materials. Where the natural subgrade is of high bearing value, such as sand or gravel, the thickness of base surface can be reduced very markedly. For example, one of the large Army bases has a sandy subgrade of many feet in thickness. A pavement composed of 4½-inch traprock

base and 2½-inch asphaltic concrete top has proved to be quite adequate. Load bearing tests on this field indicate a value under the 30-inch plate (which corresponds to 70,000 pounds wheel loading) of over 175 pounds per square inch to produce the 0.2-inch deflection. A deflection of 0.2 inch is taken as critical, but actually is conservative for sand subgrades. The service load is about 75 pounds per square inch.

There are, however, practical considerations which limit the depth of base and surface and which are tied in with construction procedure. Thus, it is not practicable, as a rule, to place a macadam in thickness much under 4 inches. Hence, the only saving that could have been made in this instance would have been ½ inch in thickness, and in that particular area the practice of constructing 4½ inch macadam bases is so well established that the familiarity of contractor and engineer with such a procedure made it highly desirable not to change. Likewise, consider the surface. On an area as big as a runway, over a mile long and 200 feet wide, there are bound to be minor settlements no matter how much care is exercised in construction. Hence, if an asphalt surface course is made less than 2 inches in thickness, there are apt to be high points in the base which would reduce the thickness of the surface to an inch or less. For these reasons, 2 inches of asphalt pavement surface or 4 inches of macadam base are the generally accepted minimum, regardless of subgrade support.

Some sands of rather uniform grain size have bearing values which cannot be raised above 50 pounds without the addition of fine material as binder. To introduce this fine material, however, for the purpose of developing greater bearing power may make the sand become frost-heaving, particularly if the ground-water level is near the surface, as on seashore locations. It is, therefore, much better to increase the thickness of the overlying base rather than to run this risk. Under such circumstances, a thickness of as much as 12 inches of macadam frequently may be desirable if stone can be obtained cheaply as it often is at seashore locations because of the possibility of water transportation of the crushed stone. These various items are mentioned because they are some of the ones which the engineer must study thoroughly and weigh carefully before arriving at his final decision as to pavement type.

131. Asphalt Pavements. Asphalt pavements for airports may be grouped in the following types:

1. Surface treatment.
2. Road mix (mixed in place).
3. Penetration.
4. Plant mix (cold-laid).
5. Plant mix (hot-mix).

There is a long list of asphalt specifications from which a selection may be made. This selection requires careful consideration by the engineer in charge of the design. Building a roadway or an airport runway is a manufacturing process and the engineer should study the methods and materials available so that the desired results may be attained at a minimum cost. Quality standard for both construction and inspection should be set up and followed very closely. Usually trouble is not the result of inadequate thickness, but rather of the failure to build density and stability into the base and sub-base with the good raw materials at hand. In other words it requires engineers to build airports just as it does to build bridges and buildings.

A flexible pavement is usually composed of four parts:

1. The *wearing course* waterproofs the surface and protects the base from abrasion and surface shear. It also increases the load-supporting value of the whole pavement.

2. The *base course* is directly beneath the wearing course and should be designed for stability, toughness, resistance to undue consolidation, and change in volume due to temperature changes and moisture.

NOTE. To prevent undue volume changes from moisture use a plasticity index of 6.0 or less. (Test fraction passing 40-mesh sieve.) To prevent undue volume change from frost action use materials which do not have more than 8 per cent passing a 200-mesh sieve.

3. The *sub-base* is the section directly beneath the base and is usually composed of fine soils. It is used to provide drainage, eliminate frost heave, and provide elasticity.

4. The *subgrade* is directly beneath the sub-base and is the native soil as it exists. NOTE. Soil surveys are necessary to determine the characteristics of the native soil.

132. Bearing Power. Asphalt pavements should be laid on a subgrade which will support the unit pressure resulting from the heaviest loads expected on the airport. As may be seen from Fig. 99 the supporting value of subgrades or foundations which underlie asphalt pavements bear a direct relation to the required thickness of compressed asphalt mixtures for runway pavements. It is, therefore, very necessary to measure the bearing power of pavements and subsoil under conditions which are the same as those experienced when the runway is put into use. It has become evident that the rating of subgrades and foundations should be determined by bearing test methods under conditions representing the soil's poorest condition and under circumstances resembling those which will exist after the surface is in place.

Soils, natural or hauled in, immediately underlying asphalt pavements should have a classification equivalent to A-3 or better. Such soils are free from the dangers of alternate freezing and thawing when proper drainage has been provided.

133. Unit Pressure. The increase in the size of planes has been accompanied by a corresponding increase in the size of tires.

The corresponding increase in the contact area of tires

on the runway distributes the load and does not increase the unit load as much as might at first be imagined.

At first thought it might be concluded that no heavier pavement is required for heavy planes than for light ones. This is incorrect. The total load for large planes is vastly greater and it has been found that stronger and heavier pavements are required for this type of traffic.

134. Unit Pressure of Bombers Same as Smaller Planes. The following statements are quoted from "Bomber Flying Fields," a publication of the Asphalt Institute.

Because of Bomber's larger tire-imprint area, it follows that its weight, when on the ground, is distributed over a wider area of a runway's surface than is the case of a smaller, lighter plane. Therefore, the *unit* pressure exerted on paved surfaces by bomber types of planes is no greater than is the *unit* pressure of smaller and lighter planes having smaller tire-imprint area. The *total* weight of bombers, however, is vastly greater than other planes, and their larger tire-imprints—while helpful—do not overcome the requirement that stronger, heavier, and more expensive pavements are needed for this type of aviation equipment than in the case of lighter planes.

This is true despite the already-stated fact, that the *unit* pressure exerted by a bomber on paved runways is actually no greater than the *unit* pressure sometimes exerted by lighter planes. It should be kept in mind that because the tire-imprints of heavy bombers are larger, bombers *do* apply correspondingly larger *total* loadings to a runway's pavement and its underlying foundation and subgrade. Comprehensive and accurate field tests have demonstrated that for any given deflection, the unit load-supporting value of a cohesive soil bears a direct relation to the *size* of the loaded area. Therefore, the rating of the unit support value of such soils is less under the tire-print area of a large bomber than in the case of a plane having tire-imprints of less area. This is so even though the *unit* pressures exerted in both cases may be the same.

This phenomenon—seemingly a paradox—concerning the behavior of soils is well understood by physicists and structural designers concerned with the problem of providing satisfactory foundations for structures of different weight and having footings of different areas. This was discussed in The Asphalt Institute's Research Series No. 8, as follows:

"The supporting value of any cohesive soil for any *unit* load varies with the area over which the total load is distributed and therefore cannot be expressed in *pounds per square inch* unless associated with some particular area. Thus, for example, a given soil may develop a 90 *pound per square inch* supporting value for one tire contact area and a 60 *pound per square inch* value for a larger tire contact area."

The Army's Corps of Engineers has long been familiar with this principle of soil mechanics. At this point it may be desirable to emphasize that a cohesive soil is the weakest type of soil, but its minimum rating can be accurately pre-determined; thus making possible the designing of adequate asphalt surfacing for any type of aircraft, *regardless of its weight.*

135. Surface Treatments. The purpose of a surface treatment is to waterproof the pavement and provide resistance to wear from the traffic. This type is seldom more than ¾ inch in thickness and is usually laid on gravel, macadam, or soil cement base courses.

136. Road Mix. The road-mix type of pavement is obtained by applying the asphalt to the aggregate which has been previously placed on the subgrade. The mixing of the aggregate and asphalt is accomplished by the use of blade graders and, after the mixing is accomplished, the mixed materials are spread and rolled in place. The macadam aggregate type is seldom laid more than 2 inches in thickness. The dense graded aggregate type may range from 2 to 3½ inches in thickness. Road mixes are not recommended for wheel loads in excess of 10,000 pounds.

137. Penetration. The penetration type of pavement is constructed by spreading the aggregate on a gravel or stone base and rolling it. The asphalt is applied by spraying asphalt from a pressure distributor. Inasmuch as 2- or 2½-inch aggregate may be used, the thickness of the pavement will be governed by these sizes. A seal coat of fine aggregate and asphalt may be used to give a fine-grained surface. This type of pavement will answer for wheel loads up to 15,000 pounds.

138. Plant Mix (Cold-Laid). This type of pavement is similar to the road mixes except that the mix is controlled by proportioning in a plant. The usual thickness is about 2 inches and is good for wheel loads up to 15,000 pounds.

139. Plant Mix (Hot-Mix). The dense graded hot-mix pavements are the highest type of asphalt pavements and are recommended for heavy loads. Careful proportioning of aggregate and asphalt takes place in the plant and the mix can be held uniform throughout the whole project.

Some plants are used as batching plants where a batch is made and mixed. It is then transported to the job in trucks.

The continuous mix is made in a special type of stationary plant where the aggregate and asphalt are mixed in a continuous flow through the mixing box. Both batch and continuous mixers are in common use not only for hot-mix types but also for cold-laid types of asphaltic pavements as well. The travel-plant type of continuous mixer is suitable only for cold-laid mixtures.

The thickness of this type of pavement ranges from 2½ inches for wheel loads up to 25,000 pounds, 4 inches for wheel loads up to 40,000 pounds, and 5 inches for wheel loads which exceed 40,000 pounds. If a thickness of more than 2½ inches is required, the pavement is usually laid in two courses.

140. Recommended Cross Sections (Runways). Airports present many of the same problems that are encountered on highway construction but often airports are more difficult because of the large area to be covered and the heavier load encountered. Widths of 150, 200, 300, and even 500 feet are involved.

The type of construction and the design of the cross section will depend upon the amount and type of traffic

which is to use the airport runways. A penetration or road-mix type may be satisfactory for an airport with light traffic but a plant-mix type should be used for heavy traffic.

The engineer who is to use asphalt construction should familiarize himself with the many different grades of asphalt available and consult the standard specifications set up by the Asphalt Institute.

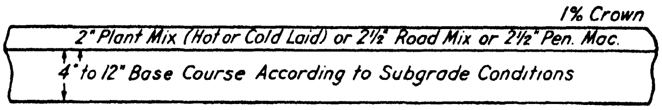


FIG. 100. Recommended sections for runways. For planes not over 10,000 pounds wheel load surface can be temporarily a surface treatment for light traffic.

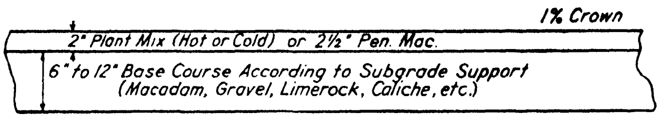


FIG. 101. For planes 10,000 to 15,000 wheel load.

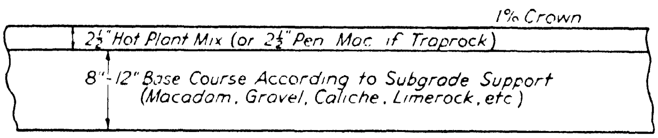


FIG. 102. For planes 15,000 to 25,000 wheel load.

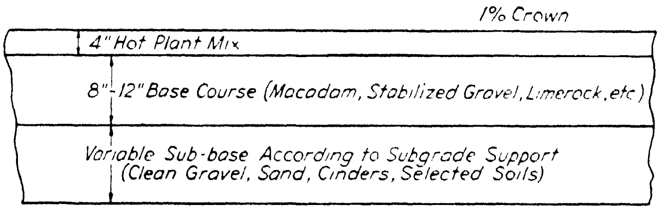


FIG. 103. For planes 25,000 to 60,000 wheel load.

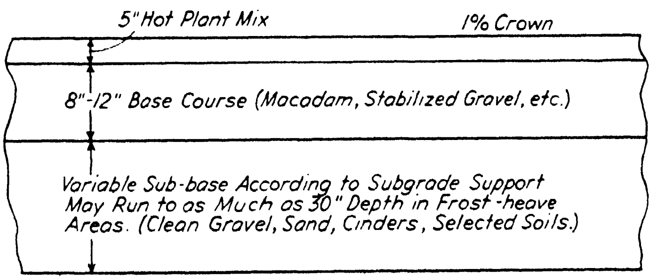


FIG. 104. For planes 60,000 to 100,000 wheel load.

141. Taxiways, Turn-Arounds, Warming-Up Areas. As stated before, the pavements which are subjected to the greatest detrimental vibration from airplanes are the taxiways, turn-arounds, and warming-up areas. At least 12 inches of surfacing is recommended for these pavements; and they should be composed of a material which is non-capillary in character. If the subgrade is clean gravel or sand no trouble will occur, but if it is composed of clay or silt it must be blanketed with a non-capillary material.

Tables XVI through XXI will serve in making estimates of the quantities of materials required in airport construction.

TABLE XVI

WEIGHT AND VOLUME RELATIONS OF ASPHALTIC MATERIALS 60° F.

Sp. Gr.	Lb. per Gal.	Gal. per Ton	Sp. Gr.	Lb. per Gal.	Gal. per Ton	Sp. Gr.	Lb. per Gal.	Gal. per Ton
0.930	7.745	258.2	0.980	8.162	245.0	1.030	8.578	233.2
0.935	7.786	256.8	0.985	8.203	243.8	1.035	8.620	232.0
0.940	7.828	255.6	0.990	8.245	242.6	1.040	8.662	230.8
0.945	7.870	254.2	0.995	8.287	241.4	1.045	8.704	229.8
0.950	7.911	252.8	1.000	8.328	240.2	1.050	8.745	228.6
0.955	7.953	251.4	1.005	8.370	239.0	1.055	8.787	227.6
0.960	7.995	250.2	1.010	8.412	237.8	1.10	9.161	218.3
0.965	8.036	248.8	1.015	8.453	236.6	1.20	9.994	200.1
0.970	8.078	247.6	1.020	8.495	235.4	1.30	10.826	184.8
0.975	8.120	246.4	1.025	8.537	234.2	1.40	11.659	171.6

Courtesy of The Asphalt Institute.

TABLE XVII

WEIGHT AND VOLUME RELATIONS OF MINERAL AGGREGATES

Broken Stone

Pounds per Cubic Yard

Kind	Sp. Gr.	Loose Spread 45% Voids	Compacted 30% Voids
Trap	2.8	2590	3300
	2.9	2680	3420
	3.0	2770	3540
	3.1	2870	3650
Granite	2.6	2400	3060
	2.7	2500	3180
	2.8	2590	3300
Limestone	2.6	2400	3060
	2.7	2500	3180
	2.8	2590	3300
Sandstone	2.4	2220	2830
	2.5	2310	2940
	2.6	2400	3060
	2.7	2500	3180

Gravel and Sand

Approximate Number of Pounds per Cubic Yard

Voids	Weight
50%	2240
45%	2460
40%	2680
35%	2910
30%	3130
25%	3350

Courtesy of The Asphalt Institute.

TABLE XVIII

GALLONS OF ASPHALTIC MATERIALS REQUIRED AT VARIOUS RATES OF APPLICATION

Gallons per 100 Linear Feet

Width ft. / Gal. per Sq. Yd.	9	12	15	16	20
0.10	10	13.3	16.7	17.8	22.2
0.15	15	20.0	25.0	26.7	33.3
0.20	20	26.7	33.3	35.6	44.4
0.25	25	33.3	41.7	44.5	55.5
0.30	30	40.0	50.0	53.4	66.6
0.35	35	46.7	58.3	62.3	77.7
0.40	40	53.3	66.7	71.2	88.8
0.45	45	60.0	75.0	80.1	99.9
0.50	50	66.7	83.4	89.0	111.1
1.25	125	166.3	208.4	222.3	277.7
2.00	200	266.7	333.4	355.6	444.4

Courtesy of The Asphalt Institute.

TABLE XIX

TONS OF MINERAL AGGREGATE REQUIRED AT VARIOUS RATES OF APPLICATION

Tons per 100 Linear Feet

Width ft. / Lb. per Sq. Yd.	9	12	15	16	20
10	0.5	0.67	0.84	0.89	1.11
15	0.75	1.0	1.25	1.33	1.67
20	1.0	1.33	1.67	1.77	2.22
25	1.25	1.67	2.08	2.22	2.78
30	1.5	2.0	2.50	2.67	3.33
35	1.75	2.33	2.92	3.11	3.89
40	2.0	2.67	3.33	3.56	4.44
45	2.25	3.0	3.75	4.0	5.0
50	2.5	3.33	4.16	4.44	5.55

Courtesy of The Asphalt Institute.

TABLE XX

APPROXIMATE QUANTITIES OF MATERIALS REQUIRED FOR ASPHALT SURFACE TREATMENTS, SURFACE COURSES, AND FOUNDATIONS AS SPECIFIED BY THE ASPHALT INSTITUTE.

Note. Items starred in Table XXI indicate that weights have been calculated upon the following basis: Specific gravity of mineral aggregate 2.65; Specific gravity of asphalt cement 1.03. If for any job the actual specific gravity varies materially from these values the following corrections may be made:

Divide weight of mineral aggregate by 2.65 and multiply by the actual specific gravity.

Divide weight of asphalt by 1.03 and multiply by the actual specific gravity. For native asphalt containing a substantial proportion of mineral matter, the proper correction should be made by subtracting the mineral matter content from weight of mineral filler and increasing the quantity of asphalt cement so that its bitumen content will equal the weight indicated for asphalt.

Asphalt Surface Treatment or Retreatment of Old Bituminous Surfaces

(Asphalt Institute Specification S-1)

Materials	Sq. Yd.	Mile 1 Ft. Wide
Cover aggregate	20.0 lb.	5.9 tons
Cutback asphalt (RC)	0.2 gal.	117 gal.

Asphalt Surface Treatment of Water-Bound Surfaces

(Asphalt Institute Specification S-2)

Materials	Sq. Yd.	Mile 1 Ft. Wide
Cover aggregate	30.0 lb.	8.8 tons
Asphalt primer	0.5 gal.	293 gal.
Cutback asphalt (RC)	0.3 gal.	176 gal.

Asphalt Surface Treatment of Loosely Bonded Surfaces

(Asphalt Institute Specification S-3)

Materials	Sq. Yd.	Mile 1 Ft. Wide
Mineral aggregate	40.0 lb.	11.7 tons
Asphaltic material (MC)	1.0 gal.	587 gal.

Courtesy of The Asphalt Institute.

TABLE XXI

ASPHALTIC CONCRETE SURFACE COURSE

(Coarse Graded Aggregate Type)

2-Inch Finished Thickness

(Asphalt Institute Specification A-2)

Materials	Pounds per Sq. Yd.	Tons per Mile 1 Ft. Wide
Coarse aggregate *	126	37.0
Fine aggregate (sand)*	65	19.1
Mineral filler *	11	3.2
Asphalt *	14	4.1

ASPHALTIC CONCRETE BASE

(Coarse Graded Aggregate Type)

3-Inch Finished Thickness

(Asphalt Institute Specification B-8)

Materials	Pounds per Sq. Yd.	Tons per Mile 1 Ft. Wide
Coarse aggregate *	204	59.9
Fine aggregate (sand)*	84	24.7
Asphalt *	17	5.0

STONE-FILLED SHEET ASPHALT SURFACE COURSE

2-Inch Finished Thickness

(Asphalt Institute Specification A-3)

Materials	Pounds per Sq. Yd.	Tons per Mile 1 Ft. Wide
Coarse aggregate *	59	17.3
Sand *	114	33.4
Mineral filler *	20.5	6.0
Asphalt *	17.5	5.1

SHEET ASPHALT BINDER AND SURFACE COURSES

Binder—1½-Inch Finished Thickness

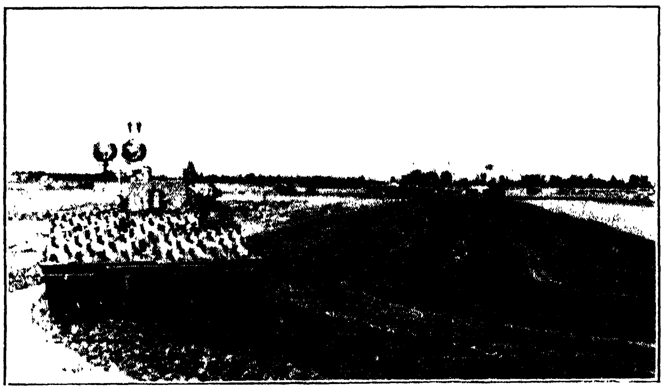
Surface Course—1½-Inch Finished Thickness

(Asphalt Institute Specification A-4)

Materials	Pounds per Sq. Yd.		Tons per Mile 1 Ft. Wide
Coarse aggregate *—binder	100		29.3
Sand *—binder	36	151	44.3
Sand *—top	115		
Mineral filler *	23		6.8
Asphalt *—binder	7	23	6.8
Asphalt *—top	16		

Courtesy of The Asphalt Institute.

142. Views of Construction Methods. Figures 105 through 119 illustrate some of the methods used in constructing asphalt runways. The subgrade should be processed carefully for proper blending of soil to uniform condition of support. The sheep's-foot roller shown in Fig. 105 is the best tool for densifying to compacted condition.

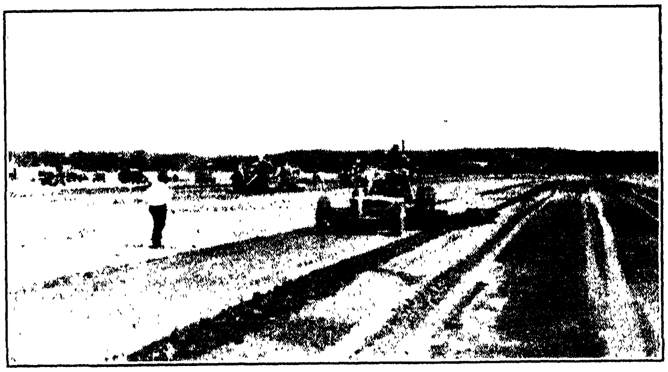


Courtesy The Asphalt Institute

FIG. 105. Sheep's-foot roller in operation.

Figure 106 shows a blade grader used in shaping the subgrade and bases to uniform crown and cross section. The rubber-tired roller is also a good piece of equipment for compaction of both subgrade and road-mix surface. (See Fig. 107.) Figure 108 illustrates the method of placing the stone base. Some forms are required so that proper cross sections will be obtained. It is important that great care be taken in this operation as the uniformity of the finished surface depends largely upon this work.

Figure 109 illustrates the method of applying hot asphalt binder to a penetration macadam base course. Figure 110 illustrates a close-up view of applying hot asphalt cement in a surface course. An application of crushed stone is used



Courtesy The Asphalt Institute

FIG. 106. Shaping with a blade grader.

to fill the voids in penetration macadam. (See Fig. 111.) In the foreground of Fig. 112 is shown a penetration macadam base; in the background an asphalt concrete surface is being placed. Hot asphaltic concrete surface courses (Fig. 113) are usually placed with a mechanical spreader. At the edge of the runway shown in Fig. 114 is placed a trench for the main storm drain at the right of the stone drain.

An important step in placing asphaltic concrete is the "setting up" of the joint, whereby it is filled so that a seamless surface results. (See Fig. 115.) The construction of hot-mix asphaltic concrete is illustrated in Fig. 116. Travel plant asphaltic mixtures (Fig. 117) are frequently employed for airport runways where light- or medium-weight planes only are present. This type of construction is particularly applicable to sand subgrades.

A typical section through an asphalt runway design for heavy plane traffic is shown in Fig. 118. In this case a particularly bad subgrade was encountered. Eleven inches of macadam base and asphalt top have been taking the heaviest traffic since 1939.

Color is a matter of choice and it is often important. Any desired color of asphalt surface can be obtained either by light seal coats or by the use of paint. The surface shown in Fig. 119 was made by using 0.25 gallon R. C.-1 and 20 pounds of white sand per square yard.

143. Description of a Model Airport. The following description of an airport runway construction will give the reader a design which has provided one of our larger airplane companies with adequate service. This airport has accommodated planes having single-tire loads up to 30,000 pounds and has suffered no deflection or deterioration by such normal use.

Courtesy The Asphalt Institute

Fig. 107. Rubber-tired roller.

Courtesy The Asphalt Institute

Fig. 108. Spreading the stone base.

Courtesy The Asphalt Institute

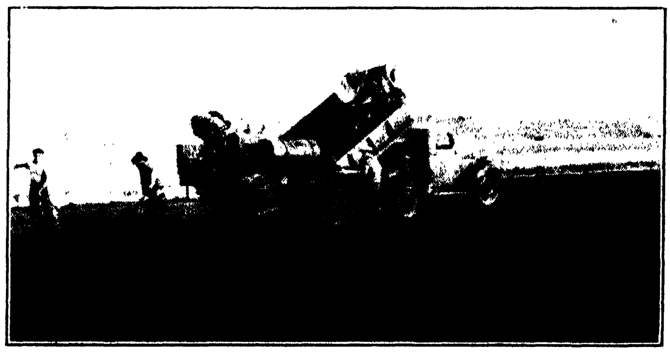
Fig. 109. Applying hot asphalt.

Courtesy The Asphalt Institute

Fig. 110. Pressure distributor applying hot asphalt.

Courtesy The Asphalt Institute

Fig. 111. Filling the voids on the surface.



Courtesy The Asphalt Institute

Fig. 112. Applying asphaltic concrete surface.

Courtesy The Asphalt Institute

Fig. 113. Mechanical spreader.

Courtesy The Asphalt Institute

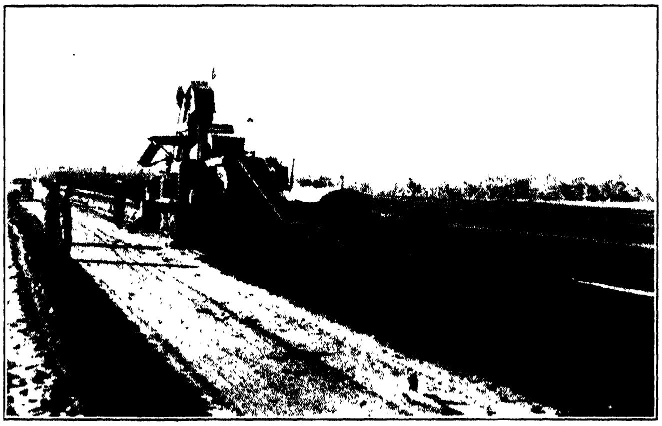
Fig. 114. Drains at the edge of the runway.

Courtesy The Asphalt Institute

FIG. 115. Joining adjacent strips

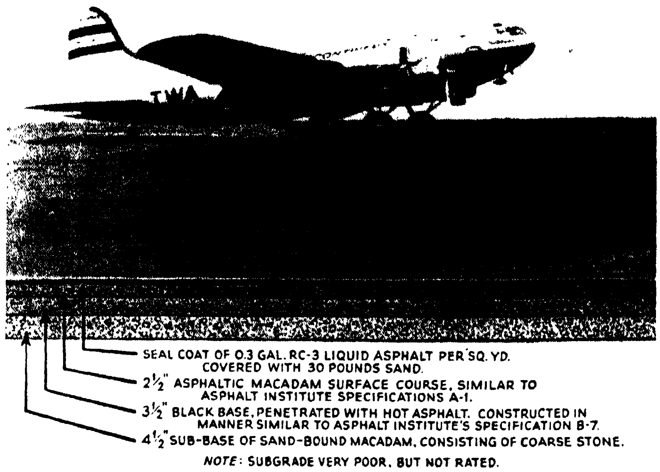
Courtesy The Asphalt Institute

FIG. 116. Laying hot-mix asphaltic concrete.



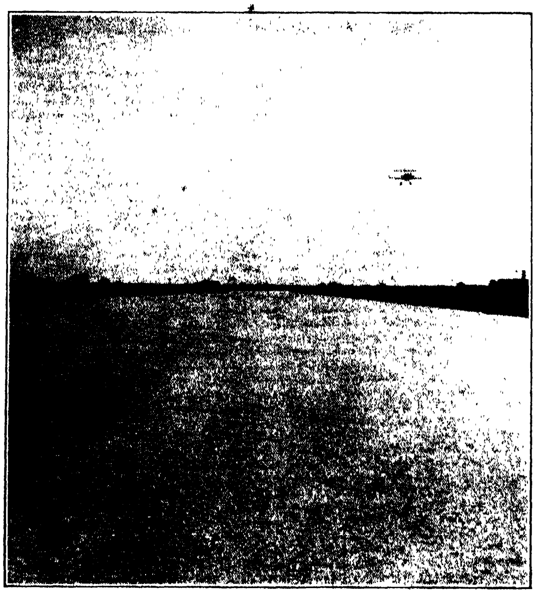
Courtesy The Asphalt Institute

FIG. 117. Traveling mixing plant.



Courtesy The Asphalt Institute

FIG. 118. Typical cross section.



Courtesy The Asphalt Institute

FIG. 119. Use of sand for coloring.

Details of Construction. Subsoil. The subsoil under the runways is of a sandy nature with a certain amount of clay interspersed. The bearing is better than average but subject to capillarity and swelling from the frost action. The water table is about 24 inches beneath the surface.

Screening Sub-Base. The sub-base course, which is 2 inches thick and composed of Trap Rock screenings, graded uniformly from 3/8 to 0 inch, cuts off the capillarity of the subsoil and gives a greatly increased and more uniform bearing for the penetration course above.

Base Course. The base course is 4 inches thick, compact measure, and consists of crushed Trap Rock, graded from 1¼ to 2½ inches, rolled, and penetrated with 1 gallon of 120 penetration asphalt. This was keyed with 50 pounds of ¾-inch Trap Rock and given a flush coat of ¼ gallon of MCl per square yard.

Wearing Course. The wearing course consists of a 1½-inch layer of hot asphaltic concrete composed entirely of Trap Rock graded from 3/8 inch to 200-mesh material and 180 penetration asphalt, each combined at a temperature of 300° and applied and rolled hot.

The outstanding results obtained from these thicknesses are due in a large measure to the quality of the stone employed, which has a Los Angeles loss of 12 (French coefficient of wear, 25); this material can be rolled with 10-ton rollers without crumbling or breakage of edges.

Therefore, the inherent stability caused by interlocking is at a maximum. Also, there are no softer fragments interspersed to start disintegration; this, in turn, would allow movement in the surrounding particles.

Greater thicknesses would have been provided if a poorer grade of material had been used.

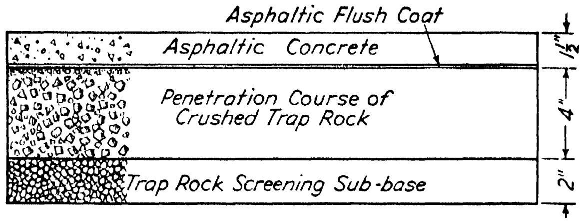


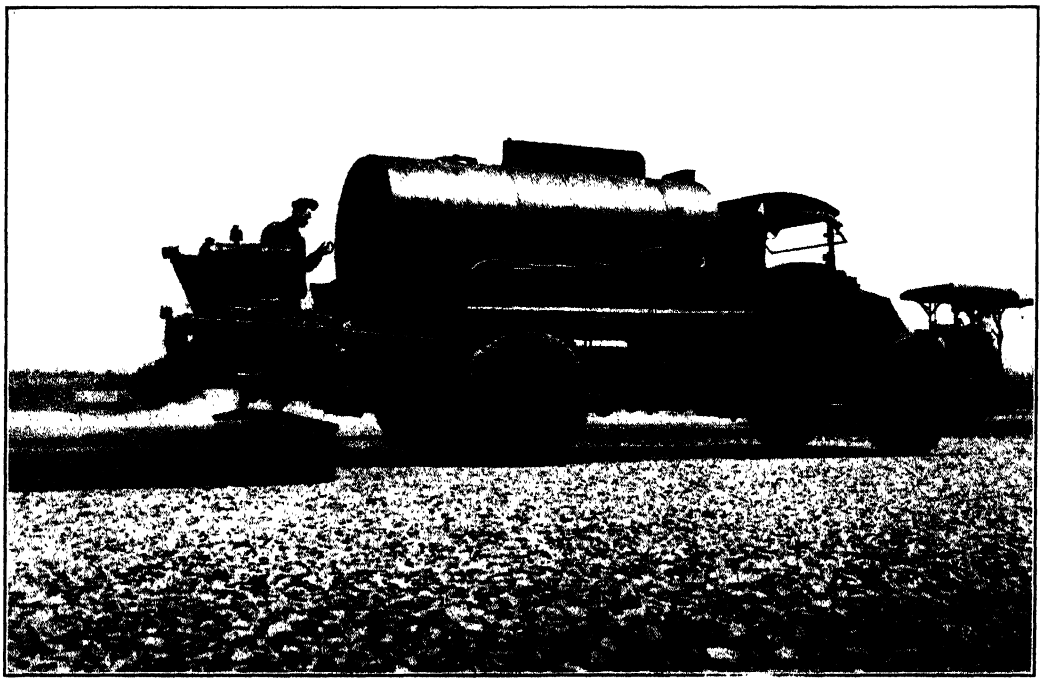
Fig. 120. Cross section of an airport pavement.

Figure 121 shows, at the lower right-hand corner, a portion of the screening sub-base rolled in place. In the foreground is the 4-inch layer of Trap Rock, graded 1¼ to 2½ inches and rolled in place.

The 1 gallon of 120 penetration asphalt is being applied to this base course, while in the background the completed runway of hot asphaltic concrete is visible.

Figure 122 shows the penetration base course after it has been penetrated with asphalt, keyed with the ¾-inch keystone, and flushed with the ¼ gallon of MCl.

The 1½-inch layer of hot asphaltic concrete is being applied to this surface through a Barber-Green asphalt spreader.



Courtesy New Haven Trap Rock Co.

Fig. 121. Applying binder by pressure distributor.

Courtesy New Haven Trap Rock Co.

FIG. 122. Blacktop paver

Chapter IX

Rigid-Type Pavements

144. Rigid Pavements. Rigid pavements are those which do not deflect or bend when the subgrade changes. Such pavements will span small depressions in the subsoil and act as a beam. One of the first theories developed for determining the thickness of a concrete rigid pavement is that known as the Older theory which was developed in 1920 by Mr. Clifford Older, who at that time was chief engineer of the Illinois State Highway Department.

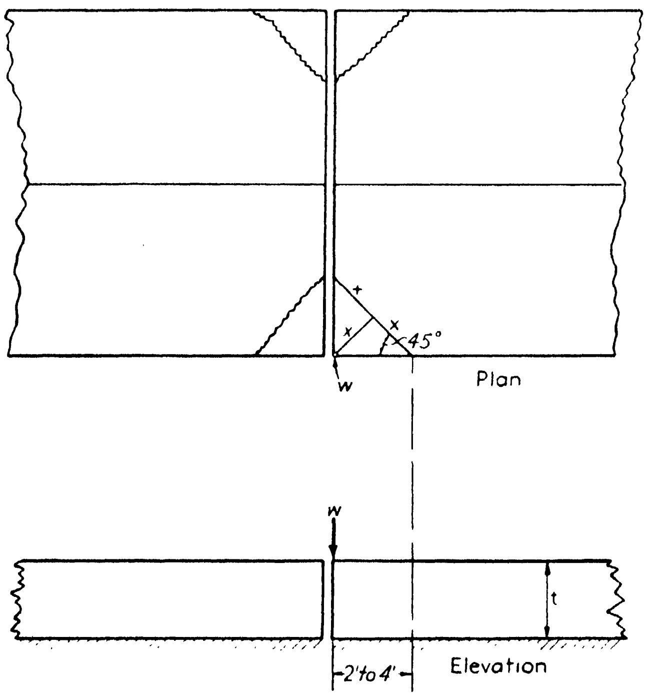


FIG. 123. Older theory.

In the investigation and tests of a large number of road slabs it was found that diagonal corner cracks developed from 2 to 4 feet from the end of the slab as shown in Fig. 123. Nearly all these cracks made a 45° angle with the edge of the road slab.

The load applied at W with the lever arm x gives an external bending moment of Wx.

The internal resisting moment $= SI/C$.

The moment of inertia $I = bd^3/12$ for the rectangular section taken along the crack in the concrete slab.

Therefore

$$b = 2x$$

$$d = t$$

then $I = 2xt^3/12$.

The distance to the neutral axis C is $t/2$.

The following equation results from equating the external moment to the internal moment:

$$Wx = \frac{SI}{c}$$

By substituting the values above, this equation becomes

$$Wx = \frac{S \cdot \dfrac{2xt^3}{12}}{\dfrac{t}{2}} = \frac{Sxt^2}{3}$$

then

$$t = \sqrt{\frac{3W}{S}}$$

This equation for the thickness of a concrete slab gives only approximate value but it is one of the fundamental theories found in most highway texts.

t = edge thickness in inches.
W = maximum wheel load plus impact.
S = allowable flexural stress in concrete in pounds per square inch.

It will be noticed that no connections such as dowels were used in the derivation of this formula. If dowels are used the load is transmitted across the corner and the for-

mula is usually modified to the following form because two slabs are acting rather than one.

$$t = \sqrt{\frac{1.5W}{S}}$$

The theory used in deriving these formulas does not consider any support from the subgrade.

Since dowels cannot be considered to operate with 100 per cent efficiency the load is probably not equally divided between two slabs that are connected by dowels. Instead of 50 per cent of the load on one slab being transmitted to the adjoining slab, it is more likely to be ⅓. The formula for thickness would then become

$$t = \sqrt{\frac{2W}{S}}$$

Example

Let the load on one wheel equal 9000 lb. and the flexural modulus equal 350 lb./sq. in. for concrete (ultimate equals 700 and safety factor equals 2).

Then

$$t = \sqrt{\frac{2 \times 9000}{350}} = \sqrt{51.5} = 7.17 \text{ in.}$$

The above example illustrates the use of the thickness formula. W, the load assumed, might answer for ordinary highway loads but airports are subjected to much heavier weights. For this reason the formula would give extremely thick pavements. It will be remembered that the bearing power of the subgrade was not considered in the derivation of this equation. It is quite evident that it should be considered for an economical design and theories have been worked out but they are quite involved for practical purposes. The Westergaard Stress Analysis is one which has received much attention in recent years. Professor H. M. Westergaard made a great many theoretical studies of the stresses developed in a concrete pavement. The application of the load was applied at three points and it was assumed that the slab acts as an integral unit in contact with the subgrade throughout. He also assumed that the subgrade reaction was vertical and directly proportional to the slab deflections. Westergaard expressed the subgrade reaction by a modulus of subgrade reaction K times the deflection at the point. The value K is expressed in pounds per square inch of deflection.

The Portland Cement Association has used the Westergaard theory to prepare a pamphlet titled "Design Data and Recommended Details for Concrete Airport Pavements."

It has been observed that the number of repetitions of loads on a pavement has much to do with the design.

There has been considerable research on fatigue behavior of concrete or its action under repetitions of stress. Some of this work has dealt with repetitions of compressive stress; other researches have investigated the behavior of concrete under repetition of flexural or bending stress.

The results of these investigations are summarized in the following statements by Frank T. Sheets, President of the Portland Cement Association.

1. When the stress does not exceed 50 per cent of the ultimate strength or when the safety factor is not less than 2, the concrete will stand a practically unlimited number of stress repetitions without failure.

2. When the stress is lower than 50 per cent of the ultimate strength or when the safety factor is greater than 2, the repetition of stress is actually beneficial and it strengthens the concrete.

3. When the stress exceeds materially 50 to 55 per cent of the ultimate strength or the safety factor is less than 2, continued repetition of stress will cause failure or breaking of the concrete.

4. When the safety factor varies between 1 and 2, the number of repetitions required to cause failure also varies, the number decreasing as the safety factor decreases.

5. When there is a period of recovery between stress applications the fatigue action is minimized.

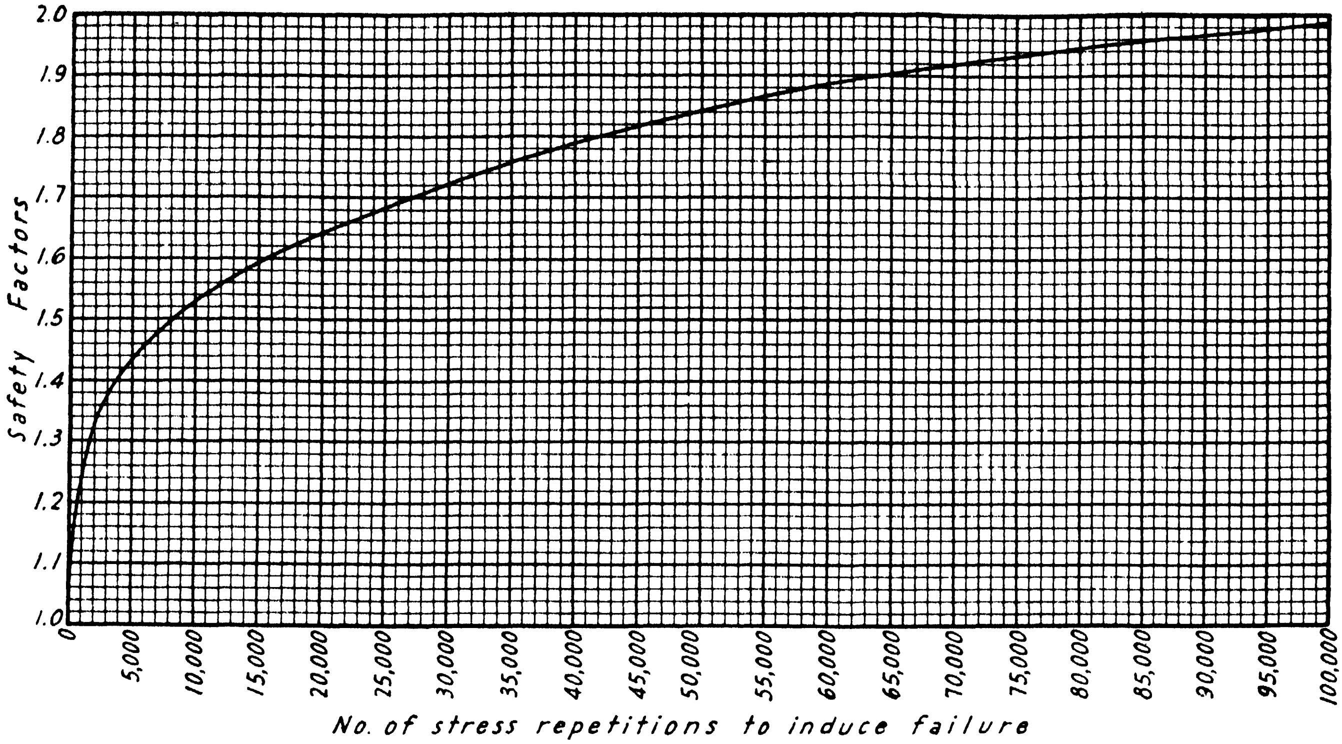
6. There is a difference in the fatigue behavior of concrete under repetitions of compressive and flexural stress, the action under flexural stress being the less severe for the same percentage of the ultimate strength.

In the design of pavement slabs the fatigue under flexural stress is the governing condition. In a pavement the compressive and tensile stresses under a given load are equal. The ultimate strength in flexure is only a small fraction of the ultimate strength in compression. Consequently when flexural stresses reach relatively high percentages of the ultimate modulus of rupture, the compression stress is only a small percentage of the ultimate strength in compression. Therefore the fatigue behavior under flexure is the governing factor in pavement design.

In Fig. 124 is shown the fatigue behavior of concrete in flexure. The relation between the safety factor and the number of stress repetitions required to induce failure of the concrete as shown represents the best data available from published reports of research work. This curve is in agreement with the Illinois Highway Department studies which are perhaps the most extensive and reliable studies available on the fatigue behavior of full-sized concrete specimens under repetitions of flexural stress.

For design purposes this curve is conservative because repetitions of stress in the tests came at relatively high frequency, whereas on a road there will almost invariably be a substantial period for recovery of the concrete between applications of the heavier loads.

The following is quoted from a memorandum on "Design Data and Recommended Details for Concrete Airport Pavements" published by the Portland Cement Association.



Courtesy Portland Cement Association

FIG. 124. The fatigue behavior of concrete in flexure.

As the safety factor is reduced below 2, the number of repetitions required to cause failure varies, decreasing as the safety factor decreases. The relation between factor of safety and number of stress repetitions to induce failure is shown in the fatigue curve [Fig. 124]. This principle has long been applied to the design of pavement for roadway and streets. It is just as applicable to the design of airport pavements as it is to street and roadway pavements. In fact, since airport pavements are usually subjected to stresses less frequently than roadways, the principle is of increased importance.

An airport of moderate size may have 150,000 sq. yd. of pavement, equivalent to that on nearly 15 miles of two-lane highway. At any one time, only one plane will normally be using this pavement. In contrast, the same area of pavement in 15 miles of roadway may have thousands of vehicles on it, many of them heavy enough to impose critical stresses on the pavement. The infrequency of stress repetition on an airport and the large area of pavement over which operations are normally distributed make it important that the fatigue principle be utilized to avoid extravagant over-design.

Consultations with airport operating officials led to the conclusion that 30 scheduled operations per hour were about all an airport could handle.

We know that on highways and railways the maximum daily traffic is less than ten times the maximum for a peak hour. So if we assume 30 scheduled operations per hour, we can count on 300 per day. Only 150 of these represent landings. As we include an allowance for landing impact in our wheel load, the stresses in taking off are less and are not likely to be critical. The 150 landings will be distributed over at least 3 runways, which is the minimum that a major airport might be expected to have. This amounts to 50 landings per runway, but because of prevailing winds one runway may have twice its share, or 100 landings; two-thirds of all traffic on one runway, with the other third divided between the other two.

Observations indicate that landings are widely distributed over a runway. Actually they come from two directions so that not all the 100 planes will land on the same end of a runway. In one series of tests conducted under the direction of the C.A.A. several hundred attempts were made to land at the same spot on a pavement. This was with a comparatively small plane which is considerably more maneuverable than a large transport or bomber. The pilot was highly skilled but he was able to hit the exact spot only about once in each hundred trials. In actual operations no attempt is made to land on the same spot. Observations and theoretical calculations, in addition to the experiment just mentioned, indicate that for 100 landings on a single runway no more than about one stress repetition will result on any one area of pavement.

One stress repetition per day equals about 11,000 stress repetitions in a thirty-year life. Referring to the fatigue curve included herein [Fig. 124] we find that 11,000 stress repetitions require the use of a safety factor of 1.54. For additional safety we have assumed a safety factor of 1.6 corresponding to about 16,000 stress repetitions. With this safety factor the pavement can handle operations up to the operating capacity of the airport.

Of course, some airports will handle many more planes than assumed here. But these will be smaller planes which will impose stresses of little or no consequence on the pavement.

A safety factor of 1.60 or more will indicate that the pavement is adequate for the normal operating capacity of the field.

145. Cross Sections. Some airport concrete sections are laid with a uniform thickness and others are built with thickened edges. The thickened edges are similar to those used in highway construction and are placed at the edges to take up the additional stress exerted at the edge by the live loads of heavy planes.

The following empirical formulas are offered by the Portland Cement Association for the computation of the thickness of a concrete slab. They represent a modification of the old corner stress formula advanced by Mr. Older and give results which closely approximate computations made by Professor Westergaard. *These formulas should be used on highway design only.* Airports are subjected to heavier loads but fewer repetitions of stresses and therefore a different method should be used. This is discussed in the latter part of this article.

The formulas for highway design follow: for protected corners (load transferred by dowels)

$$d = \sqrt{\frac{1.92Wc}{S}}$$

for unprotected corners (no load transfer)

$$d = \sqrt{\frac{2.4Wc}{S}}$$

The values of d shown in the above formulas refer to a uniform thickness. When translated into use with thickened edged formulas, it refers to the average pavement thickness along the diagonal line cut off across the slab corner as designated by line xx in Fig. 123, page 82.

In the formulas W = the load in pounds; c = the subgrade bearing coefficient; S = allowable flexural stress in the concrete.

Values of c

Bearing Power lb./sq. in.	Value of c To Use in Formula
5	1.096
10	1.000
20	0.900
30	0.842
40	0.800
50	0.770

Example

Assume a maximum wheel load of 11,200 lb.
Subgrade bearing power = 20 lb. per sq. in.
Ultimate flexural modulus = 700 with a safety factor of 2. Then

$$d = \sqrt{\frac{1.92Wc}{S}} = \sqrt{\frac{1.92 \times 11{,}200 \times 0.9}{350}}$$

d = 7.42 in. or 7.5 in. (uniform thickness)

Since the edges and corners of a slab are critical points, it may be desirable to use thickened edges as shown in Fig. 125.

The following formulas may be used to find the dimensions t_i and t_e from the value of d.

$$t_i = 0.85d$$

$$t_i = 0.85 \times 7.5 = 6.37 \text{ or } 6\tfrac{1}{2} \text{ in.}$$

$$t_e = 1.275d$$

$$t_e = 1.275 \times 7.5 = 9.55 \text{ or } 9\tfrac{1}{2} \text{ in.}$$

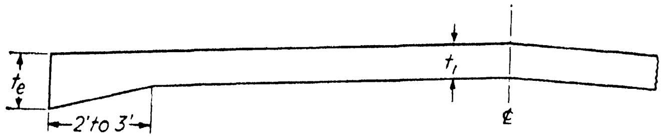


Fig. 125.

It will be remembered that a safety factor of 2 was used in the computation for the thickness of a highway slab.

If heavier loads such as those applied to airports are used a very thick slab will result. Since the repetition of loads

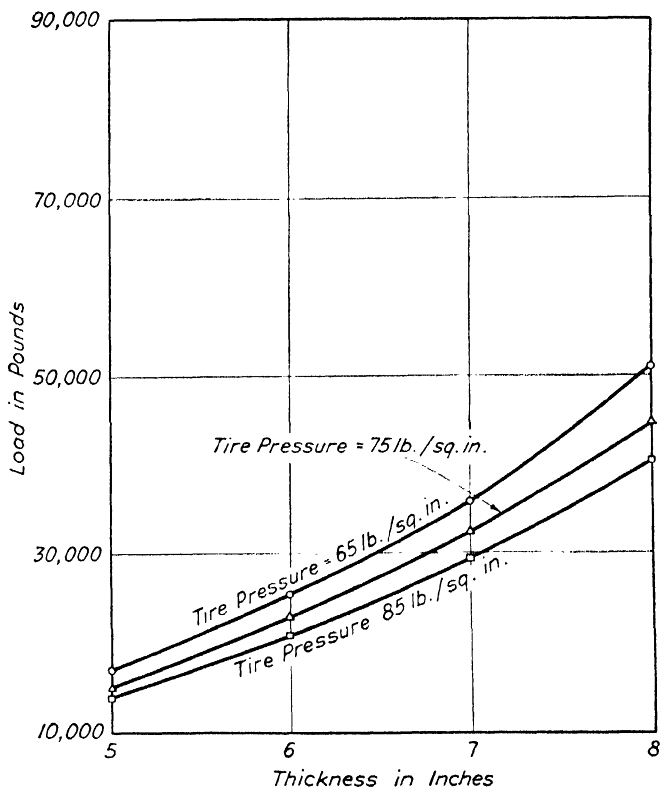


Fig. 126. Thickness of concrete runways. (For bearing value of 10 pounds per square inch and modulus of rupture of 700 pounds per square inch.)

at a point on an airport runway are fewer, a safety factor of 1.6 is considered ample for the operating capacity of the runway and is suitable for an operating capacity of 100 planes per day.

The diagrams in Figs. 126 through 130 were compiled from tables prepared by the Portland Cement Association

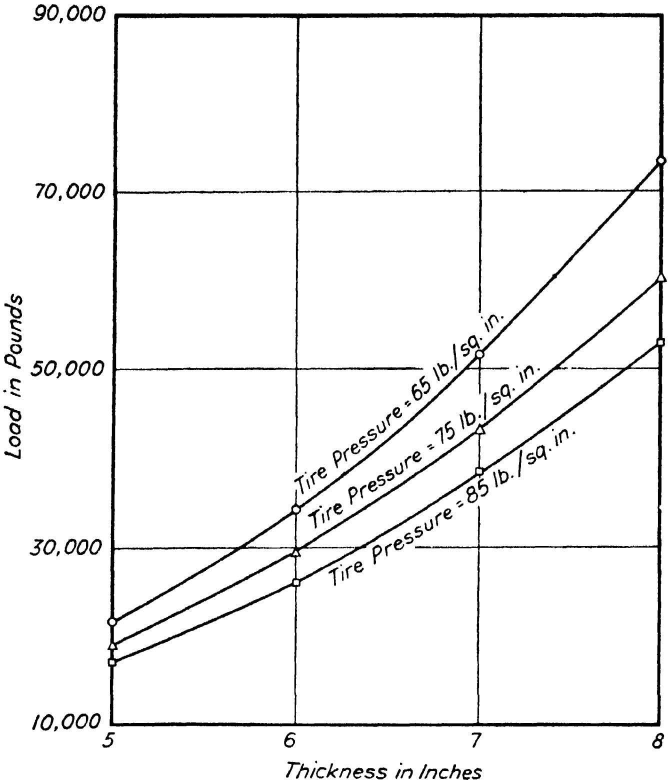


FIG. 127. Thickness of concrete runways. (For bearing value of 20 pounds per square inch and modulus of rupture of 700 pounds per square inch.)

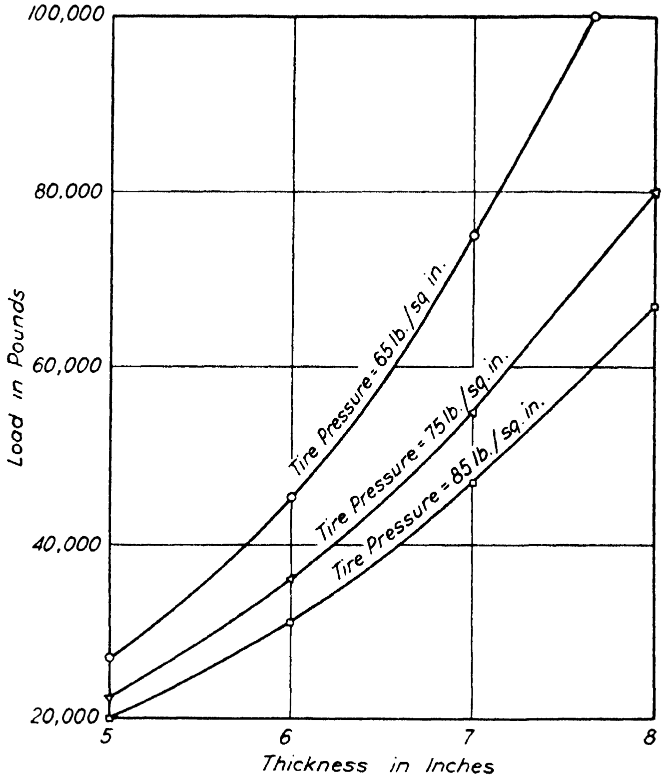


FIG. 128. Thickness of concrete runways. (For bearing value of 30 pounds per square inch and modulus of rupture of 700 pounds per square inch.)

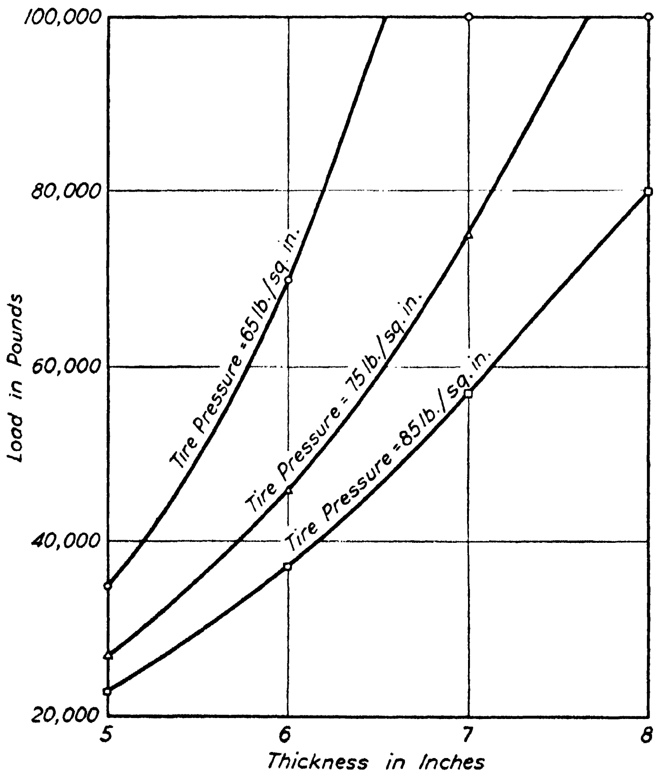


FIG. 129. Thickness of concrete runways. (For bearing value of 40 pounds per square inch and modulus of rupture of 700 pounds per square inch.)

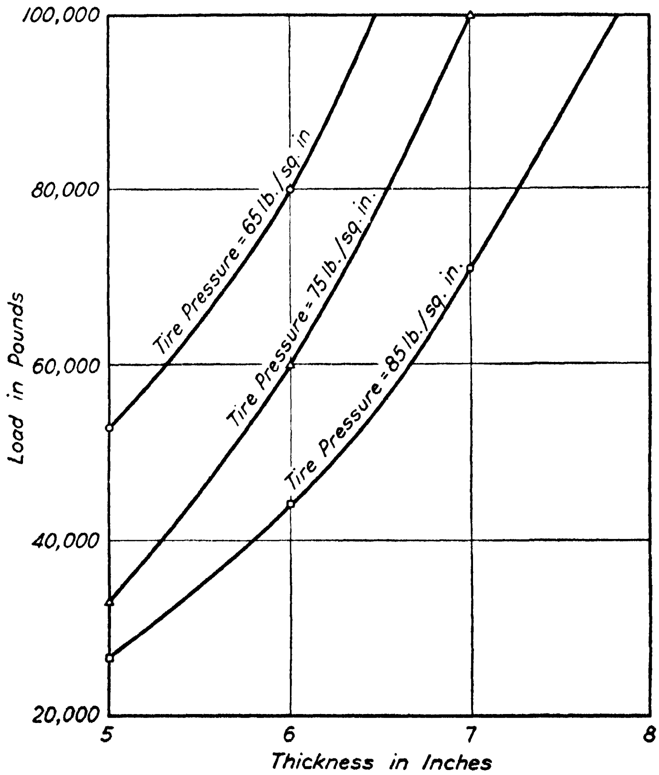


FIG. 130. Thickness of concrete runways. (For bearing value of 50 pounds per square inch and modulus of rupture of 700 pounds per square inch.)

and based on the Westergaard theory; they give interior thicknesses of slabs (values of h) in inches for various loads, bearing power of subgrade, and tire pressures.

Slab thickness determined by Westergaard is for the slab interior. It is designated as h and corresponds with t_i (interior thickness) used elsewhere in this text.

146. Spacing of Joints. Concrete will form cracks at varying intervals so that a dummy joint is advisable at intervals of from 15 to 25 feet depending on the types and grading of the coarse aggregate. Transverse expansion joints, completely through the slab, are placed at intervals of 80 to 200 or more feet and the joints are filled as in highway paving.

A longitudinal expansion joint may or may not be placed at the center of the paved area. This will depend on the width. For pavements with widths of 150 feet or less, longitudinal expansion joints may be omitted. When the width exceeds 150 feet, an expansion joint should be used. Construction joints are those which mark the edge of construction at the time of installing the pavement units.

The following is quoted from a memorandum on "Design Data and Recommended Details for Concrete Airport Pavements."

Steel-Free Designs. The sketches [Figs. 131 to 136 inclusive] are recommended for use under normal conditions when a small amount of steel is available for tie bars and dowels.

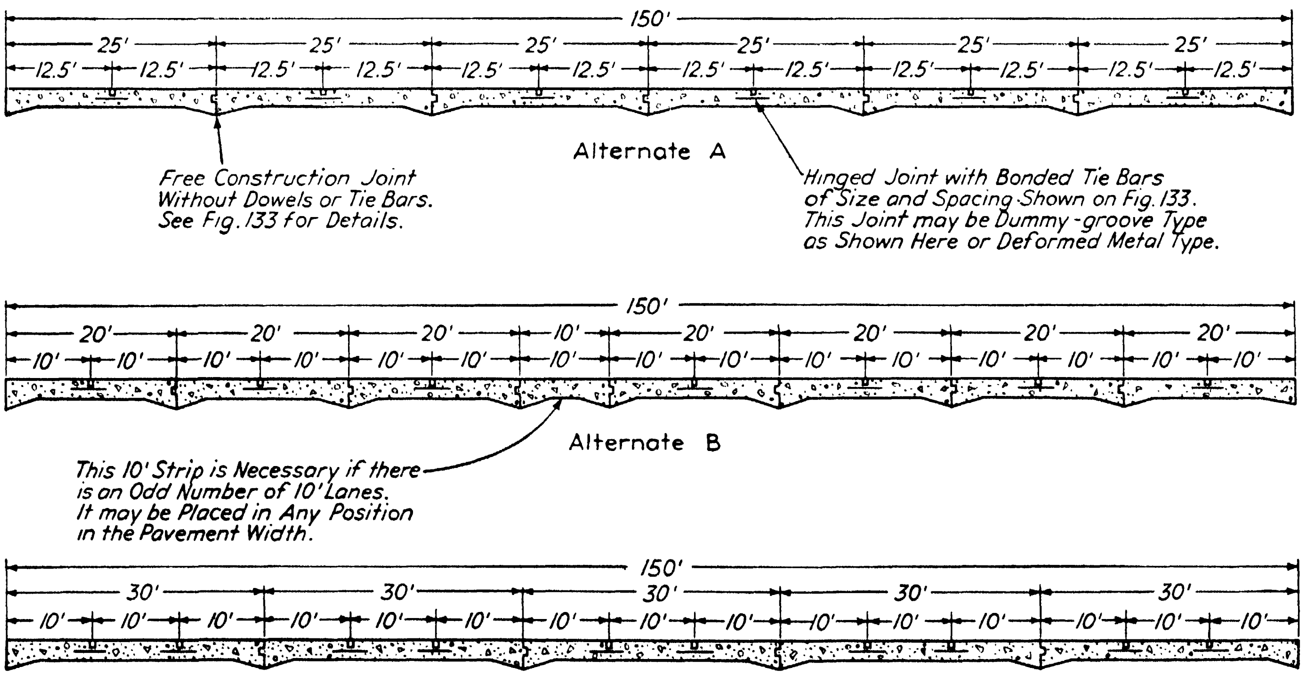
Under emergency conditions, when even this small amount of steel is not available, the designs shown in the following sketches [Figs. 137 to 141] may be substituted. Both thickened edge and uniform thickness designs are shown, with the relation between thickness of comparable designs given in sketch [Fig. 139].

With jointing as recommended, the omission of all steel calls for additional thickness of concrete only

1. At all transverse expansion or construction joints.
2. In Alternate I [Fig. 137], the thickened edge design, at those longitudinal joints at which the slab was not thickened in the designs shown in sketches [Figs. 131 to 136].

Most of the area of the pavement remains at the same thickness as would be used if the designs shown in sketches [Figs. 131 to 136] were used. The additional volume of concrete required for Alternate I design without steel [Fig. 137] shown in sketches [Figs. 137 to 141] over that required for pavement of the same structural capacity with steel dowels and tie bars as shown in sketches [Figs. 131 to 136] will range from less than 1 per cent to approximately 7.7 per cent. For Alternate II [Fig. 137], the uniform thickness designs, approximately 7 per cent more concrete is required than for Alternate I designs of the same structural capacity.

Because of this extra volume of concrete required the designs shown in sketches [Figs. 137 to 141] are slightly more costly than the corresponding designs shown in sketches [Figs. 131 to 136]. However, they are of equal value structurally and concrete pavement built according to these designs is still lower in first cost than any other type of pavement of equal structural capacity which can be constructed during the emergency when steel is not available for designs shown in sketches [Figs. 131 to 136].



Alternate C

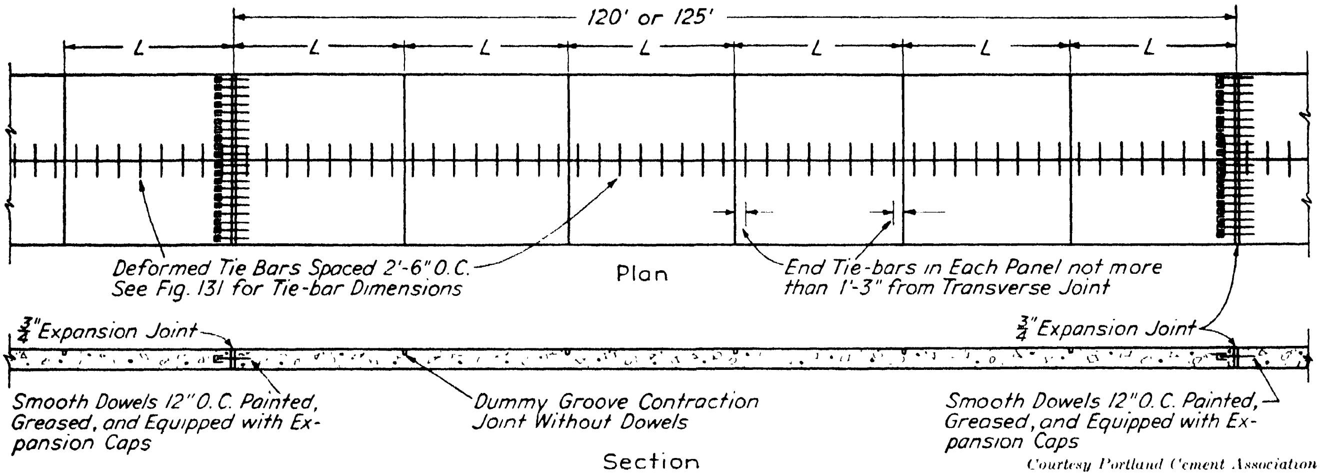
Courtesy Portland Cement Association

Alternates *A*, *B*, and *C* are of equal merit. The choice is solely on the basis of relative convenience and economy of construction and on available equipment. Notes on joint types on sketch of alternate *A* apply also to alternates *B* and *C*.

Combinations of any two of the alternates may be used to make up any required width of pavement.

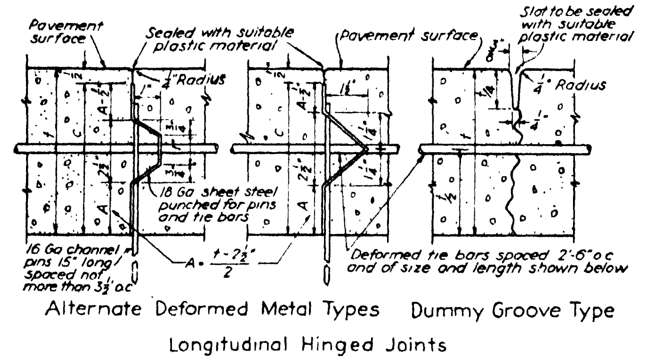
Widths greater than 150 ft. to be extension of either of the above alternate with ¾ in. longitudinal expansion joints at intervals of 100 to 150 ft.

FIG. 131.

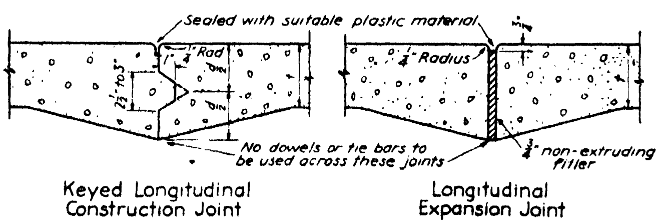


Courtesy Portland Cement Association

FIG. 132.

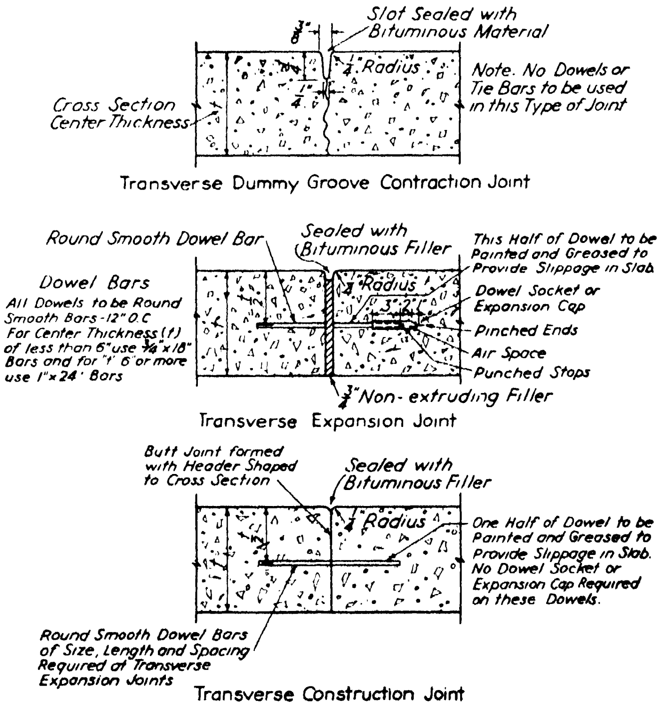


Size of Deformed Tie Bars							
For 10' Lane Widths				For 12.5' Lane Widths			
Cross Section Center Thickness		Size of Tie Bar		Cross Section Center Thickness		Size of Tie Bar	
Over	Up to & Including	Dia	Length	Over	Up to & Including	Dia	Length
-	5½"	½"	24"	-	6"	½"	32"
5½"	7"	½"	30"	6"	7"	⅝"	30"
7"	7½"	½"	32"	7"	8½"	⅝"	36"
7½"	9"	⅝"	30"	8½"	9"	⅝"	38"



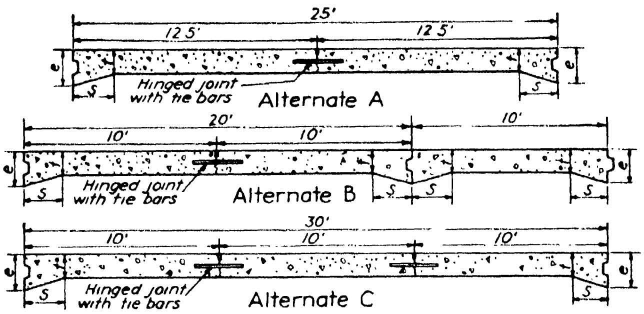
Courtesy Portland Cement Association

FIG. 133.



Courtesy Portland Cement Association

FIG. 134.



The values of t, e, and S are independent of whether A, B, or C is used
Value of t depends on loadings and subgrade supporting power
For each value of t, the edge thickness e depends upon the distance S, in which the edge thickness tapers to center thickness t
Ratios of t to e for different values of S are given in the table below

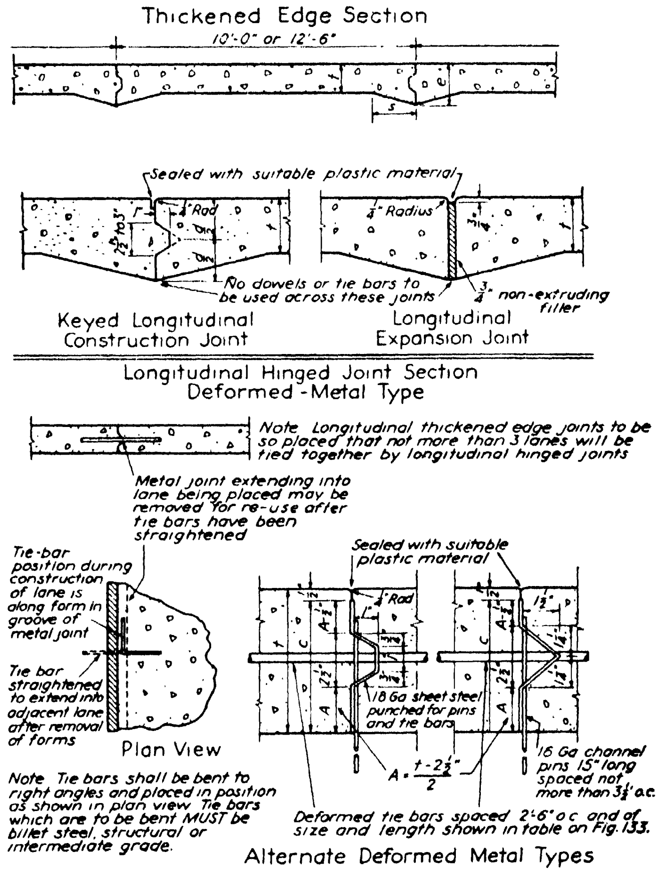
Ratio of t to e

Value of S	2'-0"	2'-2"	2'-4"	2'-6"	2'-8"	2'-10"	3'-0"
Value of Ratio e/t	1.50	1.46	1.43	1.40	1.38	1.35	1.33

There is little reduction effected in edge thickness e through use of S over 3'-0"

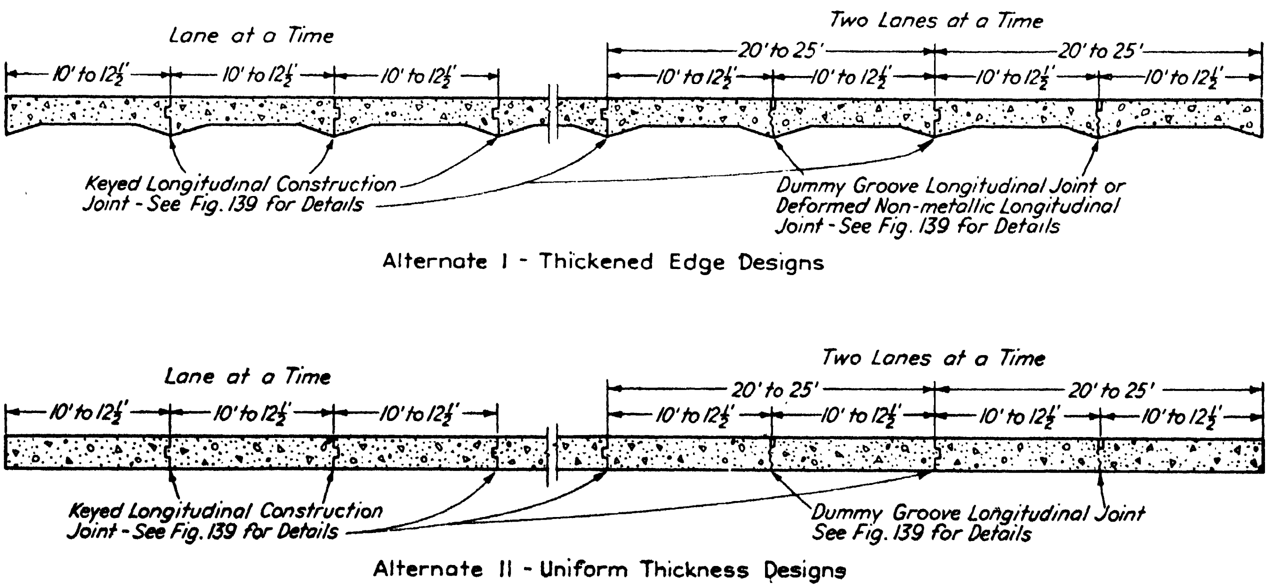
Courtesy Portland Cement Association

FIG. 135.



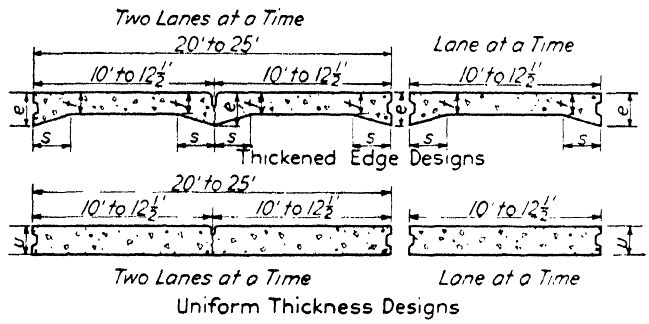
Courtesy Portland Cement Association

FIG. 136.



Courtesy Portland Cement Association

FIG. 137.



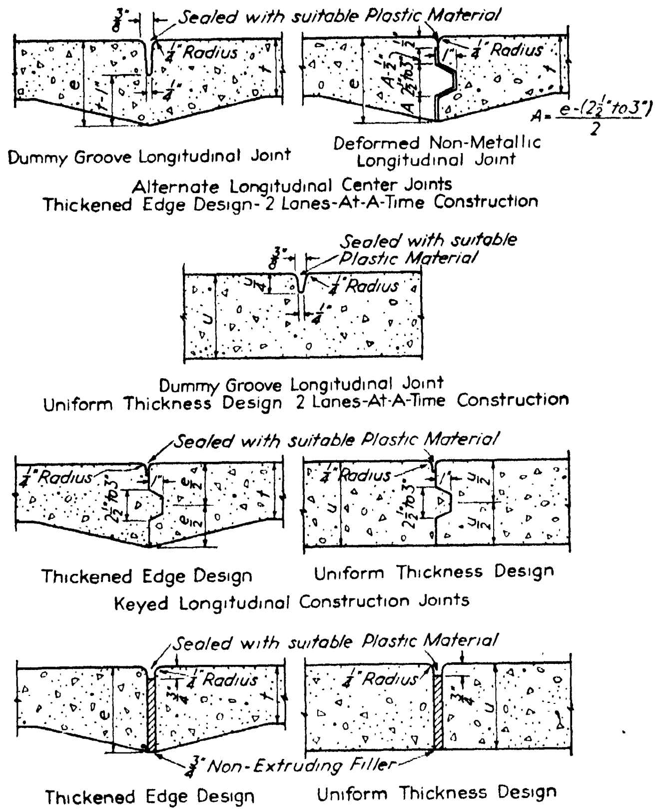
Value of t Depends on Loadings and Subgrade Supporting Power. Thickness of Equivalent Uniform Thickness Pavement u = 1.18t For Each Value of t, the Edge Thickness e Depends Upon the Distance s, in which the Edge Thickness Taper to Center Thickness t. Ratios of e to t for Different Values of s are given in the Following Table.

Value of s	2'-0"	2'-2"	2'-4"	2'-6"	2'-8"	2'-10"	3'-0"
Value of Ratio e/t	1.50	1.46	1.43	1.40	1.38	1.35	1.33

There is Little Reduction Effected in Edge Thickness e Through Use of s Over 3'-0".

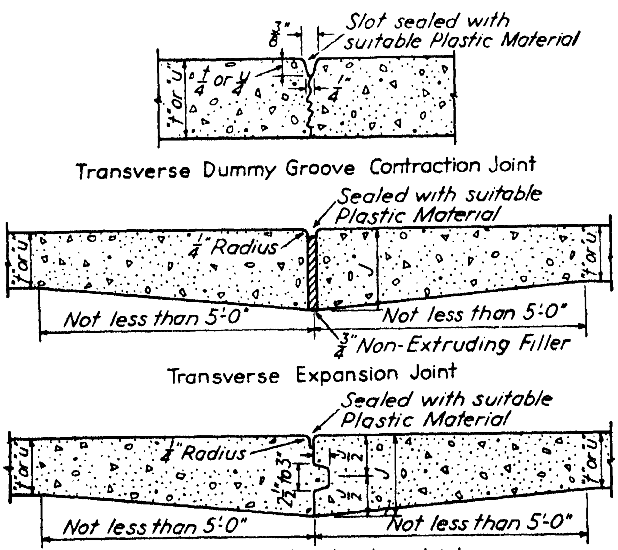
Courtesy Portland Cement Association

FIG. 138.



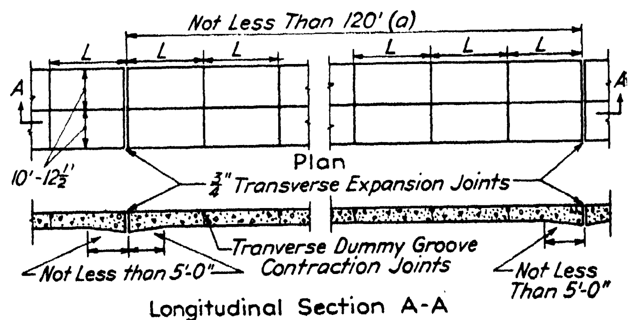
Courtesy Portland Cement Association

FIG. 139.



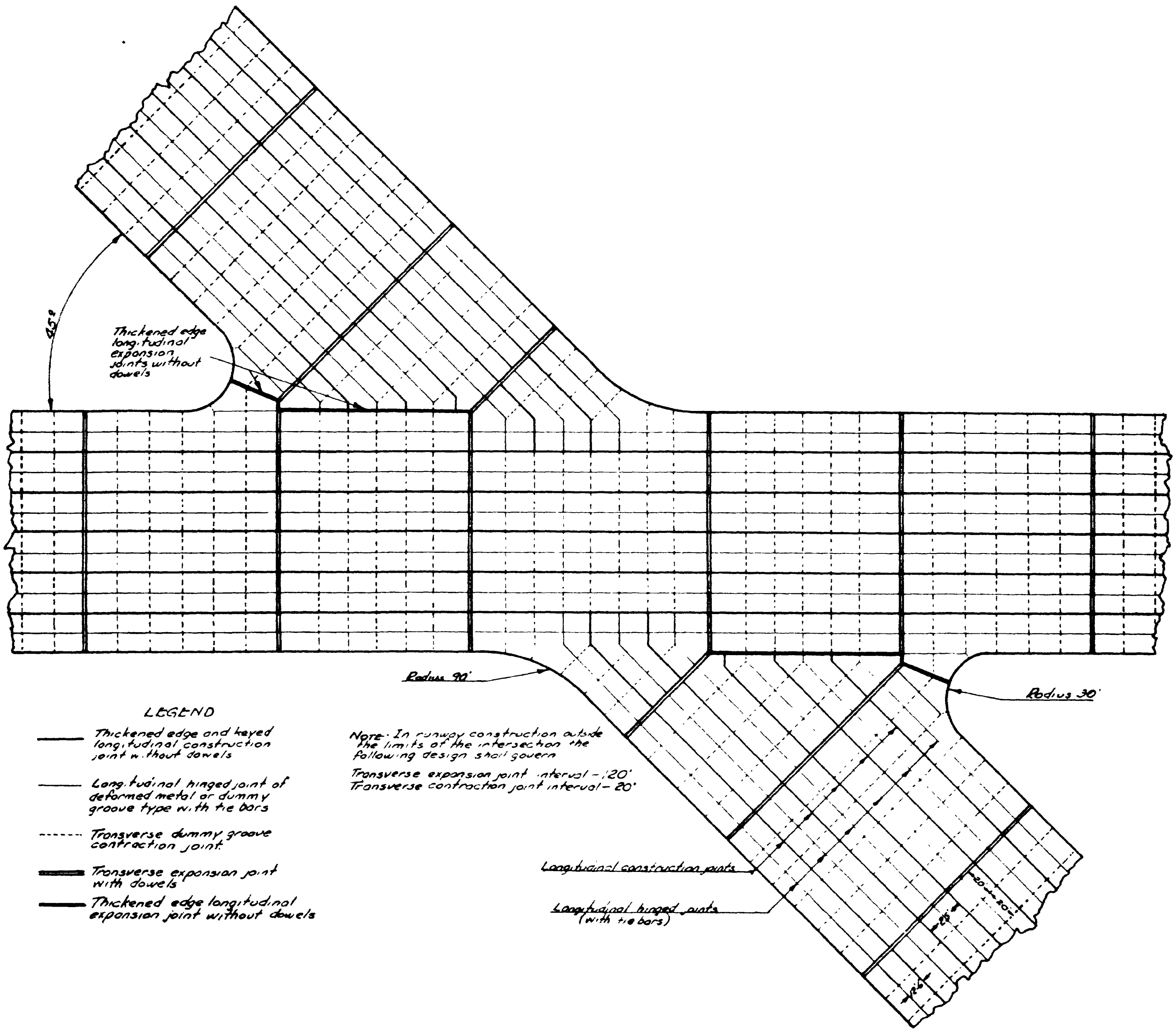
Courtesy Portland Cement Association

FIG. 140.



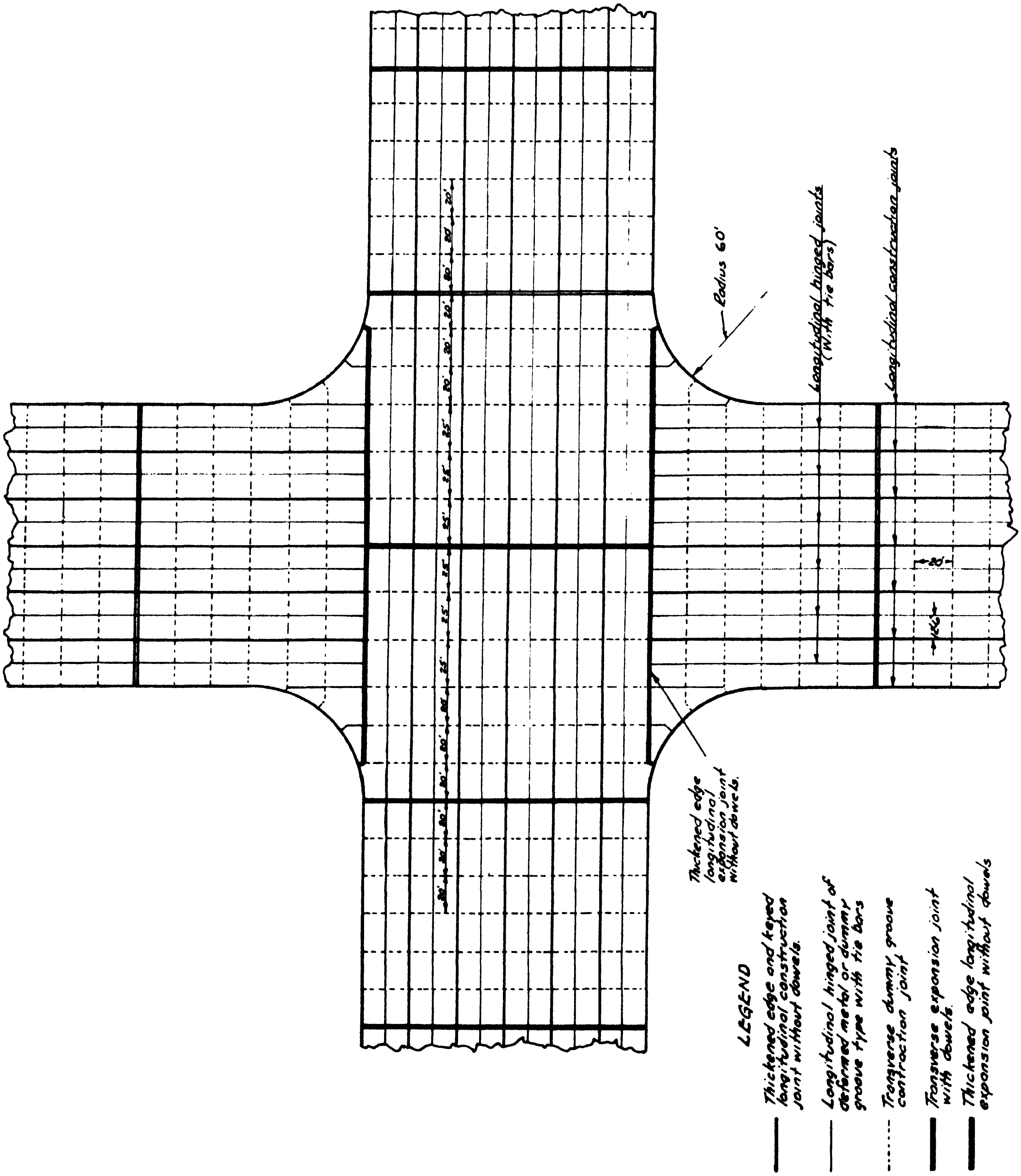
Courtesy Portland Cement Association

FIG. 141.



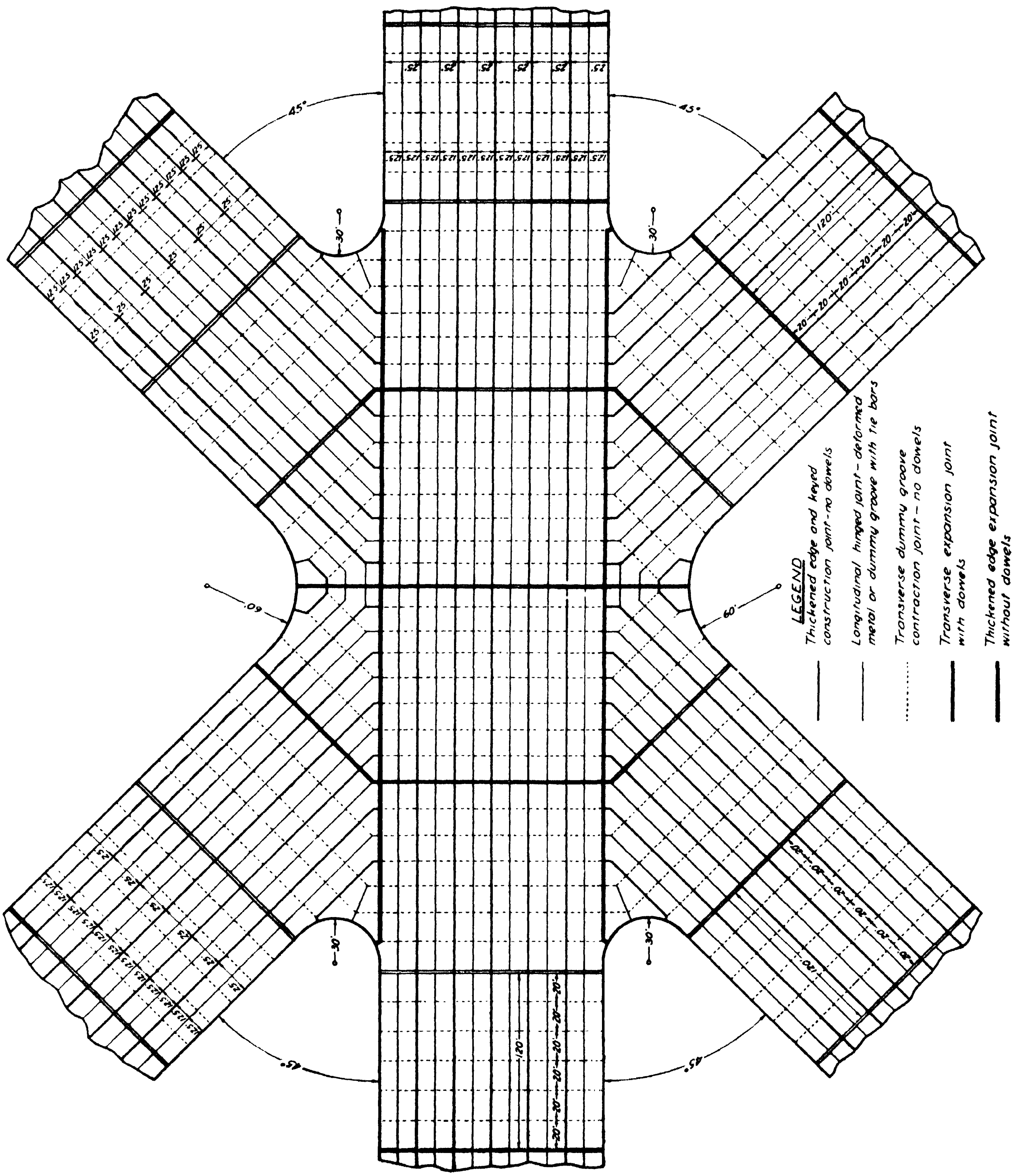
Courtesy Portland Cement Association

FIG. 142. Typical plan for jointing at intersection of two airport runways intersecting at angle of 45°. Each runway 150 feet wide.



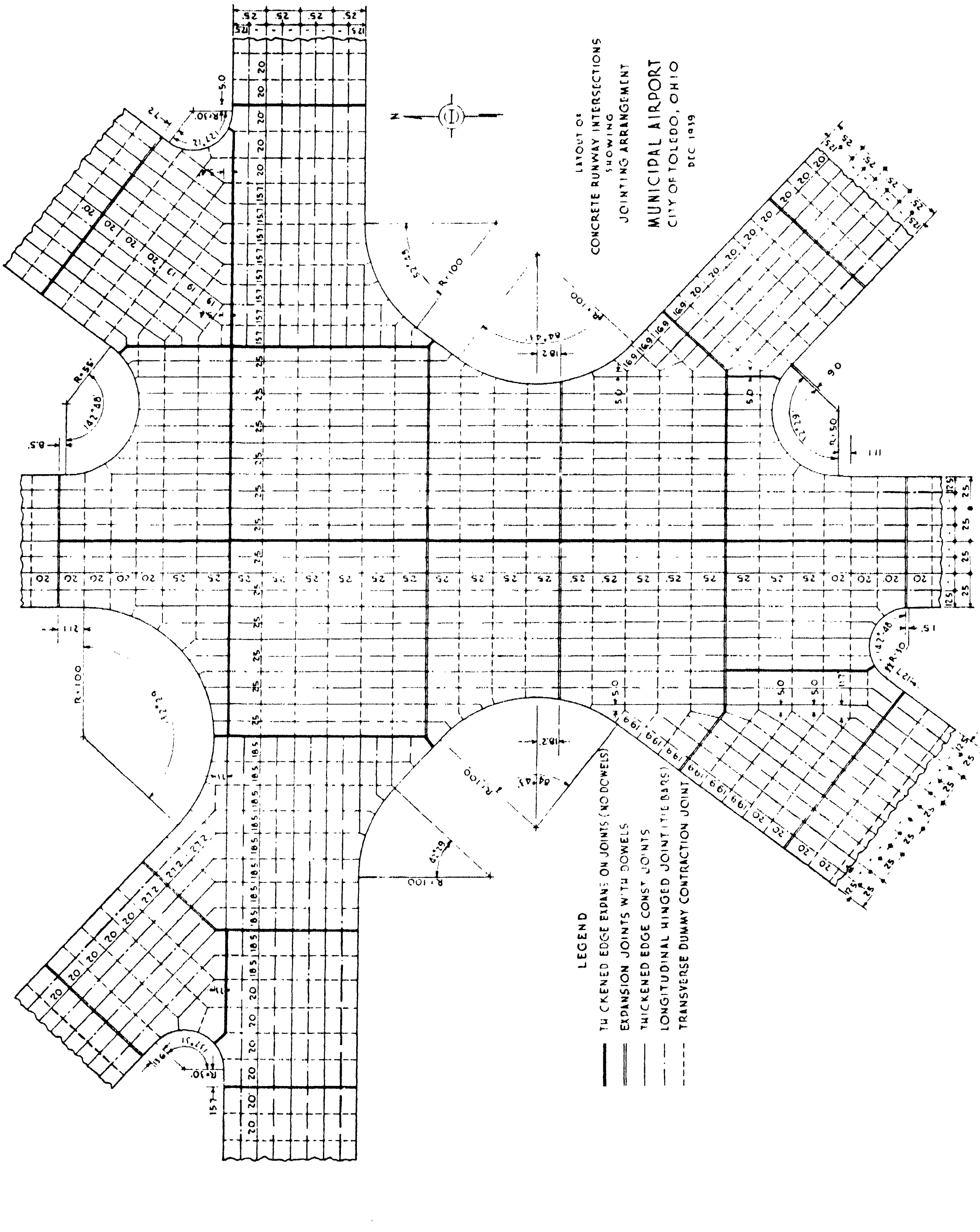
Courtesy Portland Cement Association

FIG. 143. Typical plan for jointing at intersection of two airport runways intersecting at right angles. Each runway 150 feet wide.



Courtesy Portland Cement Association

FIG. 144. Typical plan for jointing at intersection of three airport runways intersecting at a common point. Each runway 150 feet wide.



Courtesy Portland Cement Association

FIG. 145.

147. Concrete Mixes. There are several recommended methods for proportioning concrete mixes but only one very popular method will be given here. A typical problem will serve to illustrate this method, known as the *absolute volume method.* The following general considerations may serve as a guide in the selection of aggregate.

1. There should be a comprehensive and accurate knowledge of the source of all material.
2. Aggregate should be sound and durable.
3. Aggregate should be stored so that it will drain out for uniform moisture content.
4. Coarse aggregate should be supplied in two sizes. Many states use 1-inch size and 1- to 2-inch size. The objection to handling these aggregates together is that segregation of aggregate takes place. About 50 per cent of each size is used in the mix on the job.

The fine aggregate should be well graded with 10 per cent minimum passing a 50-mesh sieve and 2 per cent passing a 100-mesh sieve.

All aggregate should be stock piled for at least 24 hours.

Most specifications specify the maximum water-cement ratio expressed as gallons of water per bag of cement: such as $W/c = 5.5$ gallons per sack of cement. Some specifications use 5.0 gallons per sack of cement.

The consistency as indicated by the slump test is usually $1\frac{1}{2}$ to $2\frac{1}{2}$ inches.

Example

Let it be assumed that a 5000-lb.-per-sq.-in. concrete is to be produced in 28 days with a cement factor of 6.25 sacks per cubic yard (some engineers specify 6.0 sacks per cubic yard). The water-cement ratio is $5\frac{1}{4}$ gallons per sack of cement. The absolute volumes are next computed.

$$\text{Cement: } \frac{6.25 \times 94}{3.1 \times 62.5} = 3.03 \text{ cu. ft.}$$

6.25 = the cement factor in sacks
94 = the weight of cement in one sack in pounds
3.1 = the specific gravity of cement
62.5 = the number of pounds of water in 1 cu. ft.

$$\text{Water: } \frac{6.25 \times 5.25}{7.5} = 4.37 \text{ cu. ft.}$$

6.25 = the cement factor in sacks
5.25 = the number of gallons per sack
7.5 = the gallons of water in 1 cu. ft.

Absolute volume of water and cement = 7.40 cu. ft.

For usual aggregates, 34 to 38 per cent of sand is recommended. Since 27 cu. ft. or 1 cu. yd. is desired, the volume of aggregate will be $27 - 7.4 = 19.6$.

The volume of sand will be 34 per cent $\times$ 19.6 = 6.664 cu. ft. (absolute volume of sand); then $19.6 - 6.664$ or 12.936 cu. ft. is the absolute volume of stone or coarse aggregate.

These absolute volumes must now be changed back to weights in pounds. The following computation is then necessary.

$$\frac{\text{Weight of sand}}{2.65 \times 62.5} = 6.664 \text{ cu. ft.}$$

$$\frac{\text{Weight of stone}}{2.65 \times 62.5} = 12.936 \text{ cu. ft.}$$

Weight of sand = $2.65 \times 62.5 \times 6.664 = 1103$ lb.

Weight of stone = $2.65 \times 62.5 \times 12.936 = 2140$ lb.

The yield from the above absolute volumes will be:

Cement	=	3.03 cu. ft.
Water	=	4.37 cu. ft.
Sand	=	6.664 cu. ft.
Stone	=	12.936 cu. ft.
Yield	=	27.000 cu. ft. or 1 cu. yd.

The amount of water is 5.25 gallons per sack of cement. Therefore $6.25 \times 5.25 = 32.8$ gallons of water.

It is always necessary to check the mix on the ground and adjust as may be necessary.

148. Photographic Views Showing Concrete Construction. The pictures in Figs. 146 through 155 illustrate some of the methods used in concrete construction for airport work. Figure 146 illustrates an important step to be followed in making good concrete. The aggregate should be stored in separate sizes and should be placed on a surface which will not contaminate it. This picture illustrates one contractor's procedure. A 2-inch planked storage area was provided for aggregates everywhere throughout the project.

The batching of aggregates and cement can be conveniently and efficiently handled at batching plants as shown in Fig. 147. The aggregate batcher is shown in the foreground and two cement batchers in the background.

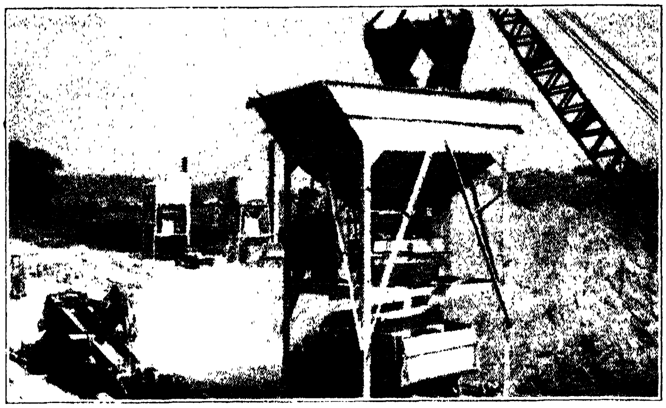
The materials from the batching plant are delivered to mixers like those in Fig. 148, which shows two 27-E pavers operating abreast on the subgrade between forms, placing the bottom 5 to 7 inches of concrete. The top 2 inches of the 9–7–9-inch section was placed with a 27-E paver operating outside the forms, as in Fig. 149.

The consistency must be maintained by careful field checks. Figure 150 shows the familiar slump test being made. The slump measures $1\frac{5}{8}$ inches in this instance. An effort is usually made to maintain a consistency represented by a slump of $1\frac{1}{2}$ to $2\frac{1}{2}$ inches.

The mixed concrete is dumped on the subgrade and spread. Figure 151 shows a Jaeger screw spreader working through three batches of concrete dumped ahead; Fig. 152 a Blaw-Knox blade spreader used in handling concrete dumped on the subgrade by the mixer; Fig. 153 a single-screed finishing machine used for the initial strike-off, the base course having been struck by a blade spreader. Figure 154 shows a second finishing machine (two-screed type) used for final transverse strike-off. Each finishing machine makes one trip. A typical Army base runway construction is shown in Fig. 155. The runway shown was built in 25-foot widths and all machines were adjusted to that width.

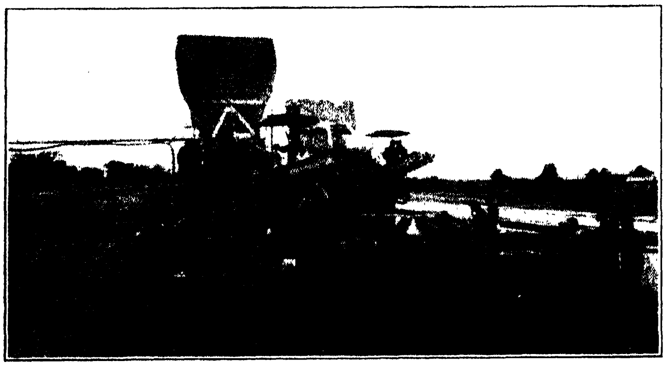
Courtesy Portland Cement Association

FIG. 146. Storage space for aggregate.



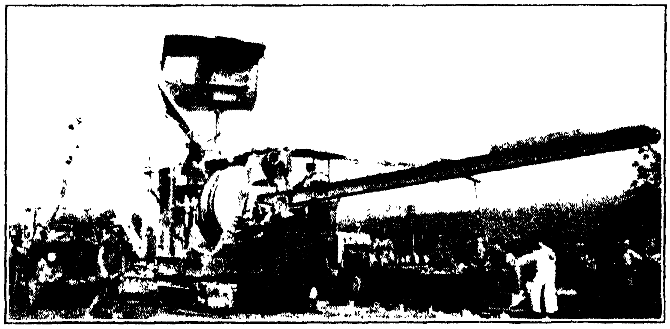
Courtesy Portland Cement Association

FIG. 147. Batching plants.



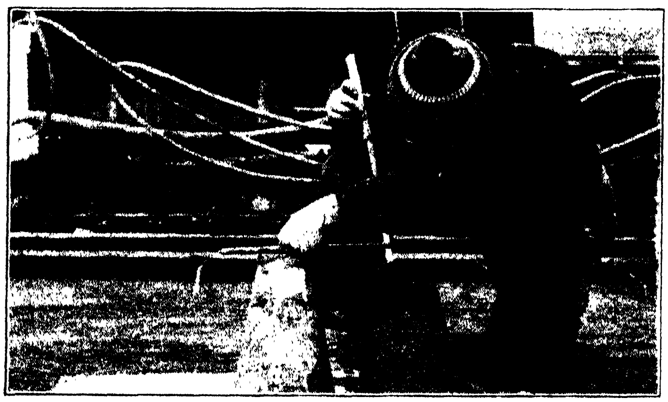
Courtesy Portland Cement Association

FIG. 148. Two 27-E pavers.



Courtesy Portland Cement Association

FIG. 149. Laying top course of a two-course pavement.

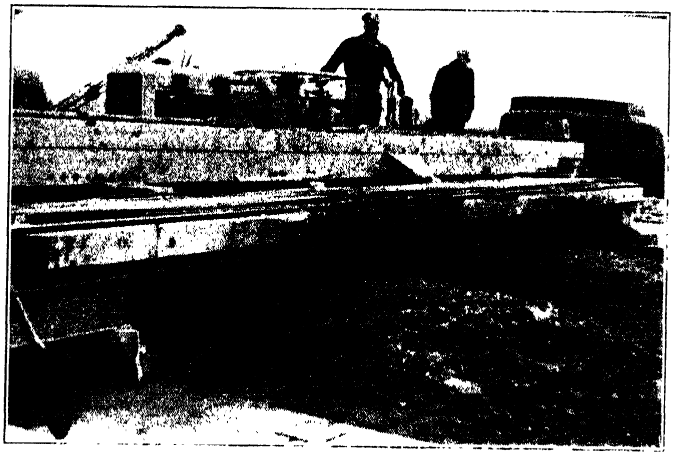


Courtesy Portland Cement Association

FIG. 150. Slump test.

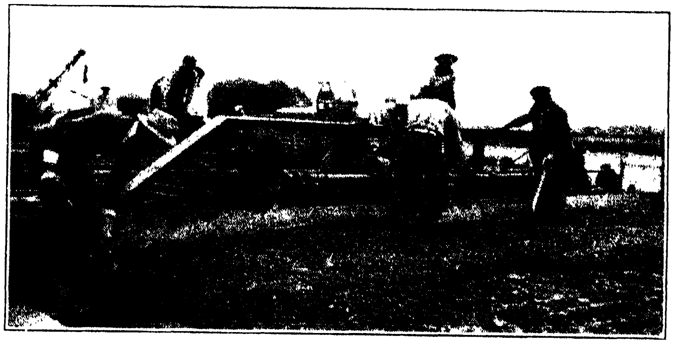
Courtesy Portland Cement Association

FIG. 151. Screw spreader.



Courtesy Portland Cement Association

FIG. 152. Blade spreader.

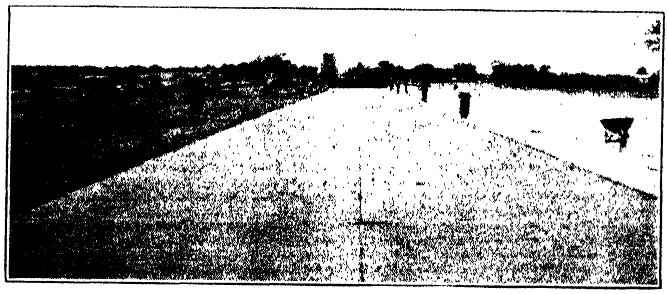


Courtesy Portland Cement Association

FIG. 153. Single-screed finishing machine.

Courtesy Portland Cement Association

FIG. 154. Two-screed finishing machine.



Courtesy Portland Cement Association

FIG. 155. Typical Army base runway.

149. Preparing the Subgrade and Setting the Forms. The preparation of the subgrade and the setting of forms is an important construction detail.

In Fig. 156 are shown drag lines being used to cast and recast top soil which has become water logged. Excavation was piled along runways, then double-cast outside the runway area to be hauled away as conditions and progress of the work permitted.

Selected material is taken from a pit and used to blend with the subgrade, thereby increasing its strength. (See Fig. 157.) The selected material is spread by means of a blade grader as shown in Fig. 158.

Figure 159 shows a crew setting forms for one side of a lane. The previously placed slab serves as the other side form.

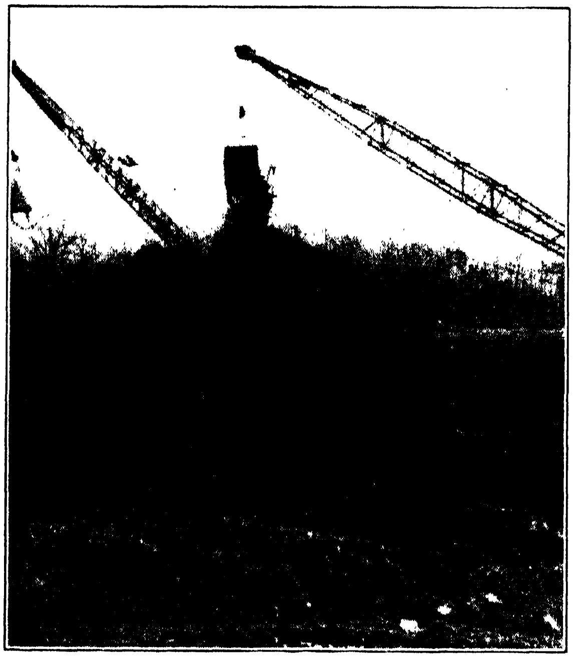
Figure 160 shows a general view of the subgrade before fine grading takes place.

Figure 161 shows a subgrader operating on the form and on the finished concrete lane. The subgrader draws a check template. A scratch template is shown in Fig. 162; it is being used to check the subgrade on 25-foot spans.

An intersection layout requires careful study of joint arrangement, as will be noted in Fig. 163. Acute angles should be avoided.

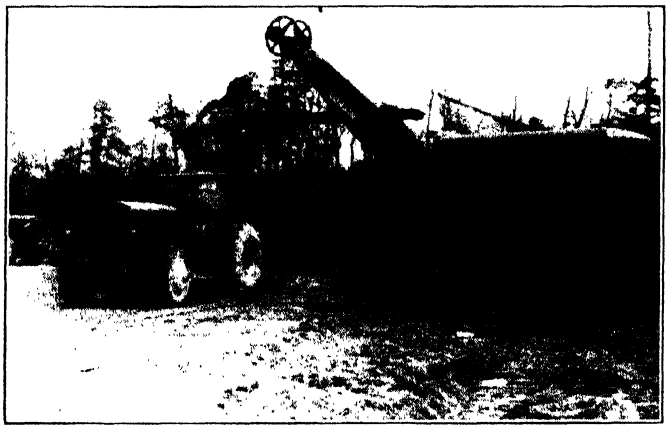
150. Dummy Contraction Joints. The formation of grooves for dummy contraction joints may be accomplished in various ways. Figures 164 through 168 illustrate one method used on an airport.

Grooves were formed by cold-rolled steel bars ¼ by 2 inches in section and 12½ feet long. In order to locate the bars directly opposite the dummy joints in adjacent lanes as well as to have them serve as a guide for the straight and vertical installation of the bars, a guide rack of two 2- by 6-inch timbers was used. The two 2- by 6-inch planks



Courtesy Portland Cement Association

FIG. 156. Use of drag lines.

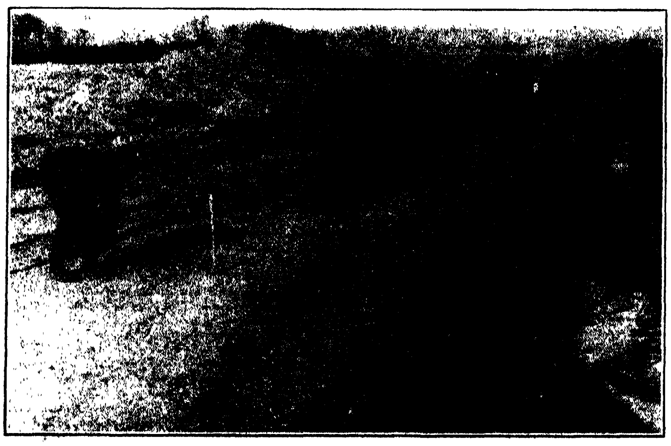


Courtesy Portland Cement Association

FIG. 157.

Courtesy Portland Cement Association

FIG. 158.

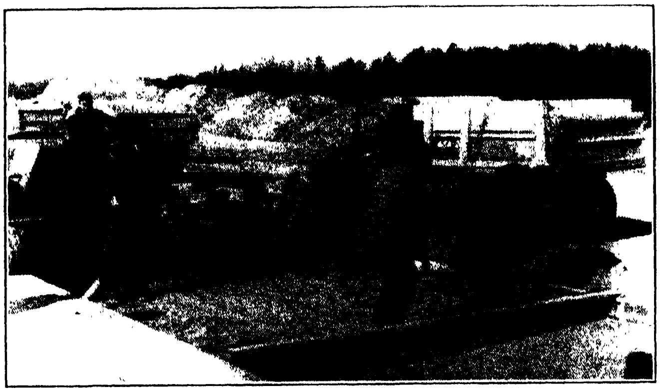


Courtesy Portland Cement Association

FIG. 159.

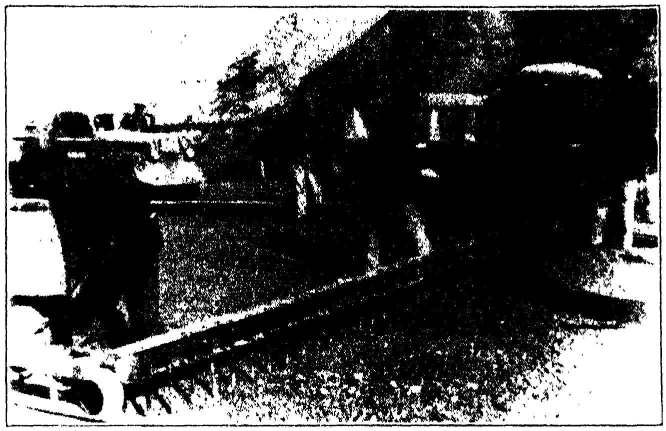
Courtesy Portland Cement Association

FIG. 160.



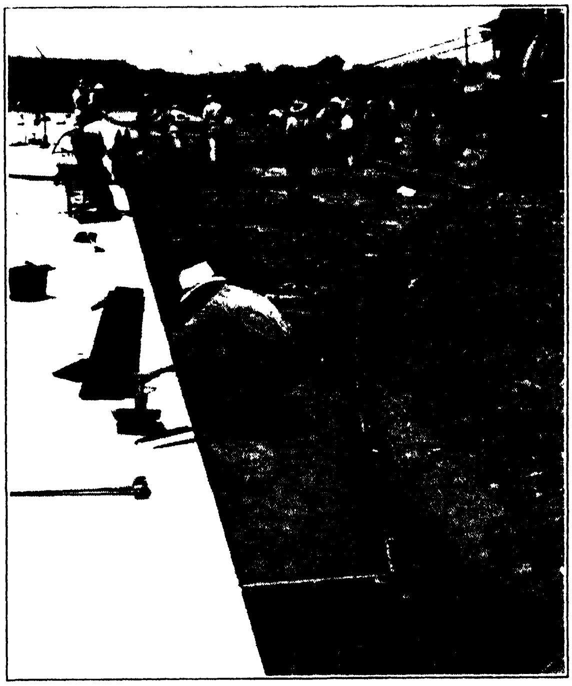
Courtesy Portland Cement Association

Fig. 161.



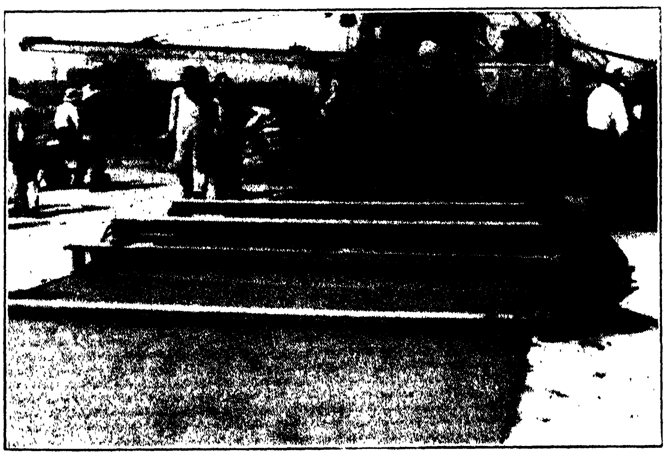
Courtesy Portland Cement Association

Fig. 162.



Courtesy Portland Cement Association

Fig. 163.

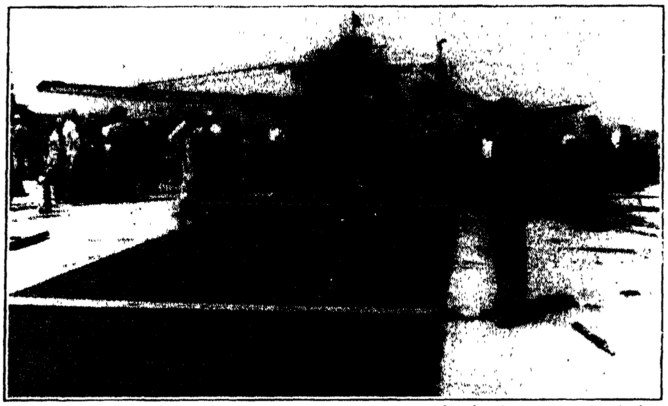


Courtesy Portland Cement Association

Fig. 164.

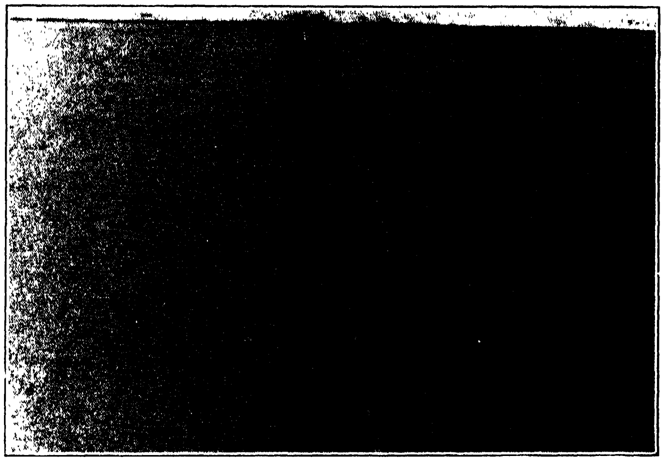
Courtesy Portland Cement Association

Fig. 165.



Courtesy Portland Cement Association

Fig. 166.



Courtesy Portland Cement Association

Fig. 167.

Courtesy Portland Cement Association

Fig. 168.

were cleated together at each end with a space left between them just wide enough to permit passing of the steel plate (see Fig. 164).

The steel plates were driven into the concrete by means of manual jackhammers, notches being cut in the planks to permit the bit on the end of the hammer ram to pass down to the level of the pavement (see Fig. 165). Note that the men stand on each end of the plank in order to hold the steel plate in line transversely as well as vertically when it is being rammed into the surface of the concrete.

When the steel plates had been driven so that the top edge was flush with the surface of the pavement, the wooden guide was moved forward to the next dummy joint position. In Fig. 166 the guide is about to be moved forward. The notches in the wood template can be clearly seen in this picture.

Figure 167 shows the steel plate in place after the template has been removed. Here small marks left by the notched bit at the end of the hammer can be seen in the pavement.

After the wooden template had been removed, finishing operations were carried on over the top of the steel plates. When the water sheen had left the pavement, the steel bars were pulled up about ¼ inch above the surface of the pavement (see Fig. 168). The joint then was edged against this projecting plate.

When the concrete hardened sufficiently to leave the groove intact, the steel plate was pulled out progressively. In Fig. 168 the plate in the foreground is being given its initial pulling and those in the background have been pulled out about halfway. When the plate can safely be removed without damaging the edges of the groove, one end is left in the groove and the workman on the opposite end pulls the plate toward him, using the end of the plate as a trowel to even up the bottom of the groove.

It is possible to start lifting the steel plates at five to ten blocks back of the mixer, or 100 to 200 feet, depending on weather conditions and the rate of hardening of the concrete. They can be completely lifted from the pavement at a distance of 200 to 300 feet back of the mixer.

Steel strips are then cleaned and oiled and carried forward for re-use. On the job described here, about 15 to 20 steel plates were able to take care of the two 27-E mixers used in laying the 9–7–9-inch slab, placed in 12½-foot strips.

151. Longitudinal and Transverse Joints. Figure 169 shows a Lakewood single-screed finishing machine followed by a Jaeger double-screed machine. Note the portable internal vibrator at work at the left. Concrete at slab edge is vibrated on many jobs.

A slab is usually straight edged, as shown in Fig. 170. This operation follows the finishing machine.

The longitudinal joints may be tied together by tie bars. Figure 171 shows the use of a split tie-bar protection plate which is removable without straightening the bars.

Figure 172 shows a tongue-and-groove longitudinal joint formed by wiring the tie-bar shield to the side forms. The view shows ⅝-inch tie bars straightened preparatory to placing the concrete pavement.

The transverse expansion joint is an important detail and Fig. 173 shows a special plate installed against the side forms for holding the ends of the joint filler securely in vertical position. Tie-bar protection shields are stopped at this plate in order to provide full slab section at the joint.

The transverse joint shown in Fig. 174 is known as the Bethlehem joint. Pins and a U-shaped metal cap maintain the ¾-inch fiber joint filler and joint assembly in perfect alignment.

Where the day's work stops at an expansion joint, a wood bulkhead may be cut to fit dowels. In Fig. 175 a 3-inch plank has been cut to fit the dowels.

The final tooling of transverse joints is accomplished by using wood straightedges resting on the pavement surface as a guide for the finishing tools. This procedure is shown in Fig. 176.

After the forms are removed the joint end plates and metal keyway plates are salvaged for re-use. The edges of the slab are clean-cut with no honeycomb. It is important that the joints be straight and plumb throughout the entire length and depth. A good example is shown in Fig. 177.

The curing of concrete should receive careful attention. One method is illustrated in Fig. 178 where burlap mats are spread and soaked with water distributed from a tank truck using perforated pipe.

152. Soil Cement. Soil-cement pavements are used on many airports. The pavement is made by thoroughly mixing cement and water with the native soil. From this statement it would appear that the process is simple but in reality it requires much technique and experience to produce the best results. The hardness, durability, soundness, and evenness of soil-cement surfaces depend upon the technique used in final finishing operations.

The soils laboratory is an important element in selecting the quantities of soil, cement, and moisture required to produce a desired mixture. Specimens with cement contents of 8, 10, and 12 per cent by volume of compacted mixture are generally selected for test. The Portland Cement Association has published a laboratory handbook titled *Soil Cement Mixtures*. This handbook, along with their publication titled *Soil Cement Roads—Construction Handbook*, presents in practical form the complete procedure for testing and construction of soil cement.

No field construction should be undertaken before field control factors required for success have been determined in the laboratory. This places all work on a foundation of sound, scientific procedure and removes the process from the hit-or-miss, trial-and-error class.

The control data determined by the laboratory consist of the cement content, moisture content, and density to be used in construction. Thus, moisture content of 14 per cent and an oven-dry weight, or density, of 112 pounds per cubic foot may be specified for a soil cement containing 10 per cent cement by volume. The specified moisture must be present at time of compaction and the soil-cement mixture compacted to the specified density. Cement content is given in terms of per cent by volume of compacted soil-cement mixture, with a bag of portland cement considered as 1 cubic foot. For example, with a cement content of 10 per cent by volume specified, 1 square yard of roadway 6 inches thick will require 0.45 bag.

In general, most soils will have a satisfactory hardness and durability with cement contents ranging from 8 to 12 per cent by volume. Corresponding optimum moistures for different soils may vary from 9 to 25 per cent and densities from 90 to 135 pounds per cubic foot oven-dry weight. Also, as a generality, the lower cement and moisture contents occur with the higher densities. In general discussions, it is common practice to use a cement content of 10 per cent by volume, an optimum moisture of 12 per cent, and a density of 115 pounds per cubic foot to describe an average construction condition.

The laboratory supplies the engineer with the data required for drawing up the plans for the soil profile. These same data are used to show the station limits of each soil-cement mixture and the corresponding cement content, moisture content, and density to be used in construction. The same data permit the estimating engineer to determine quantities of cement and water required together with the square yards of construction or "processing." Likewise, the same data are used by the specification engineer to draw up special provisions covering cement, moisture, and density requirements and the type and weights of sheep's-foot rollers and smooth tandem rollers needed for construction.

The cost of soil cement is less than that of concrete but on the other hand it should not be compared with concrete as to durability and compressive strength. The compressive strength will vary with the type of soils used and may

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Fig. 169.

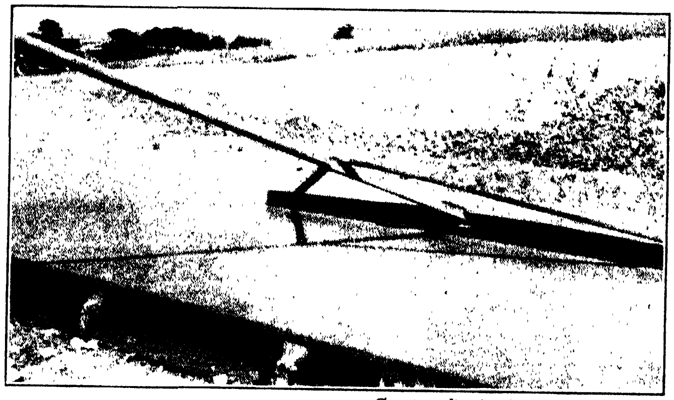

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Fig. 170.

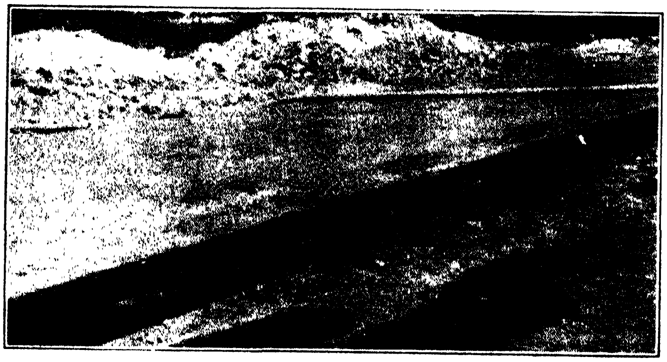

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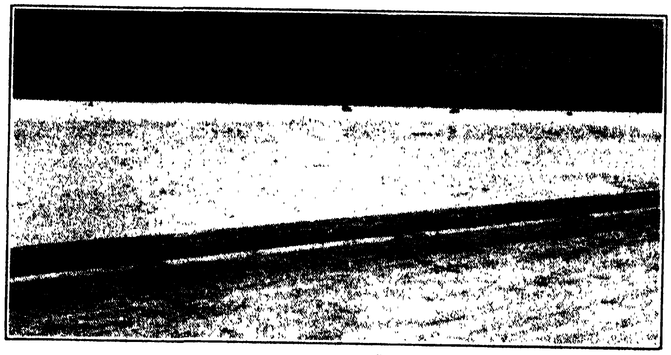
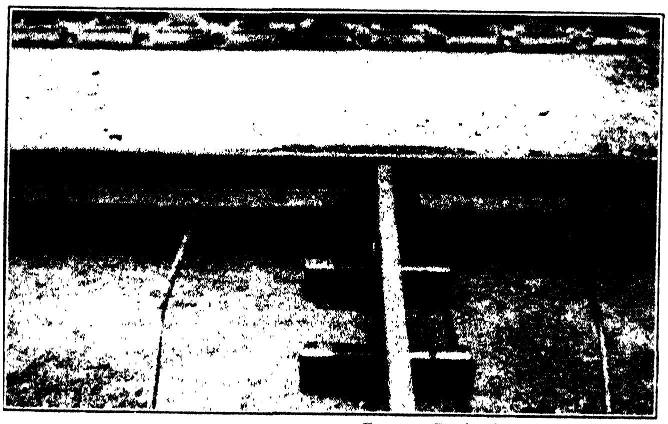
Fig. 171.



Courtesy Portland Cement Association

Fig. 172.



Courtesy Portland Cement Association

Fig. 173.

Courtesy Portland Cement Association

Fig. 174.

Courtesy Portland Cement Association

FIG. 175.

Courtesy Portland Cement Association

FIG. 176.

Courtesy Portland Cement Association

FIG. 177.

Courtesy Portland Cement Association

FIG. 178.

Courtesy Portland Cement Association

FIG. 179. Panorama from intersection of two runways.

be anything from 350 pounds per square inch for clays and loam to 2000 pounds per square inch for sand when tests are made on 28-day-old specimens.

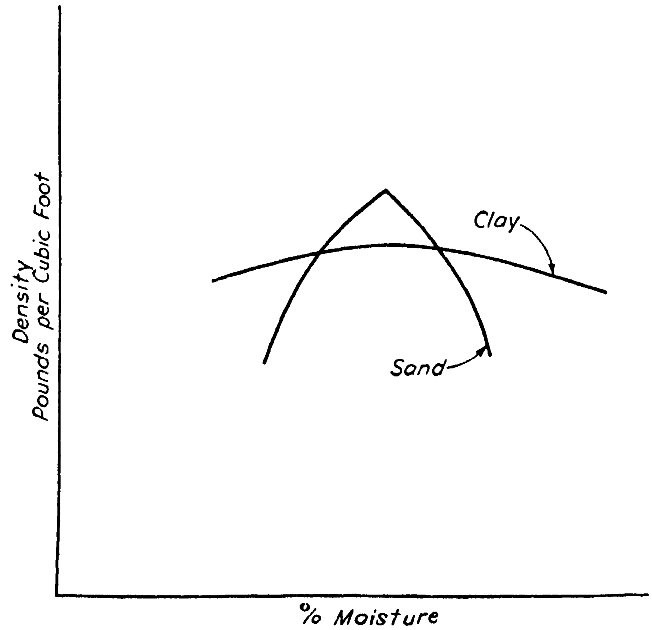


Fig. 180. Typical moisture-density curves for clay and sand.

Soils in the A horizon may contain large percentages of organic matter and that is not good for soil-cement mixtures. They would require a large amount of cement and they might be uneconomical. The B horizon is better but it may not be constant and therefore the cement content must be varied; this involves a close check on the control. The C horizon is quite constant and usually causes less trouble in treatment. The importance of a good soil map is therefore of great importance.

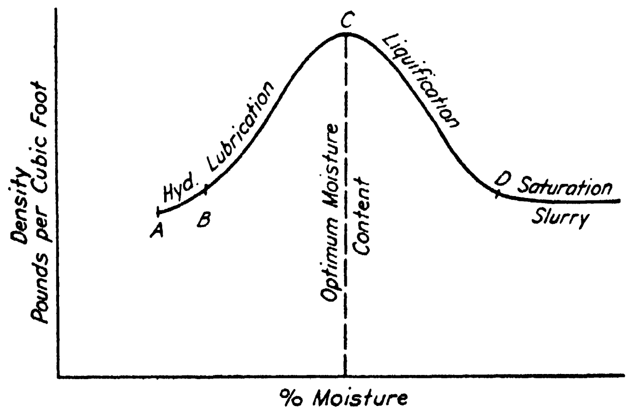


Fig. 181. Stages through which soil passes as percentage of moisture is increased.

The Proctor moisture-density test is of importance in soil-cement construction as the greatest density is attained at optimum moisture content and gives best results. The moisture-density curve illustrated in Fig. 180 has some significant points which should be kept in mind.

Generally the curve for sand soils will be narrow and rather pointed; curves for clay will usually be flat.

Another significant fact to keep in mind is the relation shown in Fig. 181. This is the general form of the moisture-density curve. The section from A to B may be termed the hydration period, B to C the lubrication period, the point C the point of optimum moisture content. The weight per cubic foot and therefore the density increase as the per cent moisture content increases. After the point of optimum moisture content is reached a liquification period takes place as the per cent moisture content increases.

During this period the weight per cubic foot decreases. This period is indicated by the section of the curve from C to D. A point D is eventually reached where the soil becomes saturated and forms a slurry.

During soil-cement construction it is important to keep the moisture content at optimum per cent content to cut down on the voids and give the greatest density.

153. Soil-Cement Construction. Soil-cement construction is seldom more than 6 inches in depth after compaction. Success in the building of this type of pavement depends upon complete coordination of all details by the construction superintendent, the resident engineer, and the soils engineer.

Construction Operations. The following is recommended by the Portland Cement Association.

At the earliest opportunity, the job personnel should review construction and engineering operations together. This joint conference of the project engineer, construction superintendent, soils engineer, timekeeper and foreman will result in a mutual understanding of operations. In general, the project engineer should be responsible for:

1. Crown, grade and line.
2. Depth and width of treatment.
3. Portland cement content.
4. Satisfactory pulverization of roadway soil.
5. Satisfactory mixture of soil and portland cement.
6. Satisfactory mixture of moisture and soil-cement.
7. Satisfactory compaction and surface finish.
8. Satisfactory cover to curtail evaporation.

The soils engineer makes tests for and is generally responsible, with the resident engineer for:

1. Pulverization and moisture control.
2. Density and depth control.

The construction superintendent is responsible for the prosecution of the work and bears with the resident engineer the responsibility of satisfactory depth and width control, pulverizing,

spreading and mixing proper cement quantities, spreading and mixing proper quantities of water, proper compaction, finishing, and cover.

The photographs in Figs. 182 to 194 illustrate the series of operations necessary in building a soil-cement surface. The first operation after grading is the spreading of the cement. Figure 182 shows one method used wherein the bags of cement are spotted and then dumped directly on the graded soil. The next operation is to mix the cement with the soil. In Fig. 183, right lane, the cement is being spread with a spike-tooth harrow. The left lane shows an equipment train dry-mixing the soil and cement. Figure 184 shows the process of dry-mixing soil and cement with heavy duty cultivators and a rotary speed mixer.

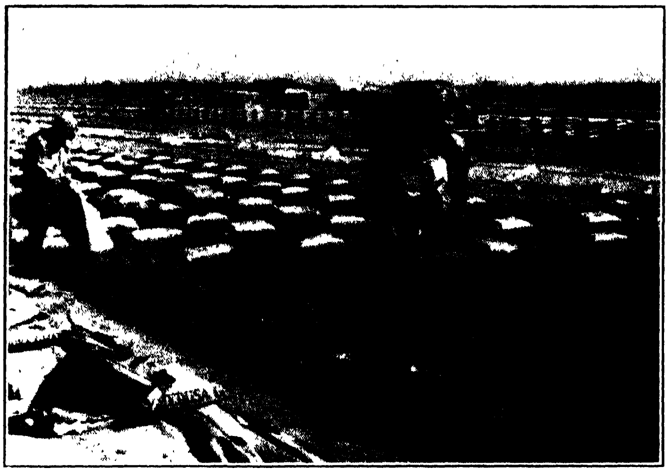
The next step is the moist-mix operation. It consists of spreading water with a distributor; this is followed immediately by an equipment train consisting of cultivators and a rotary mixer. Figure 185 illustrates this operation.

The edges are cleaned out by the use of a single-bottom plow equipped with a small form blade, edge plate, and depth guard. See Fig. 186. The edge mixture is next thrown in by auto patrol for thorough mixing with the harrow which follows immediately. See Fig. 187.

The next operation after mixing consists in compacting the soil-cement mixture from the bottom with sheep's-foot rollers as shown in Fig. 188.

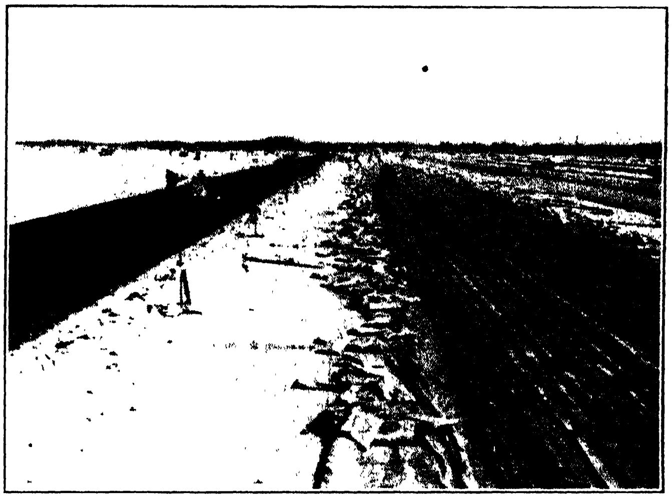
The runway must next be shaped by means of a blade grader. A motor blade grader is used for this operation, as shown in Fig. 189.

The surface of the runway is next slightly roughened by means of nail drags and broom drag. A smooth-surfaced mulch is created which is next rolled with a tandem steel



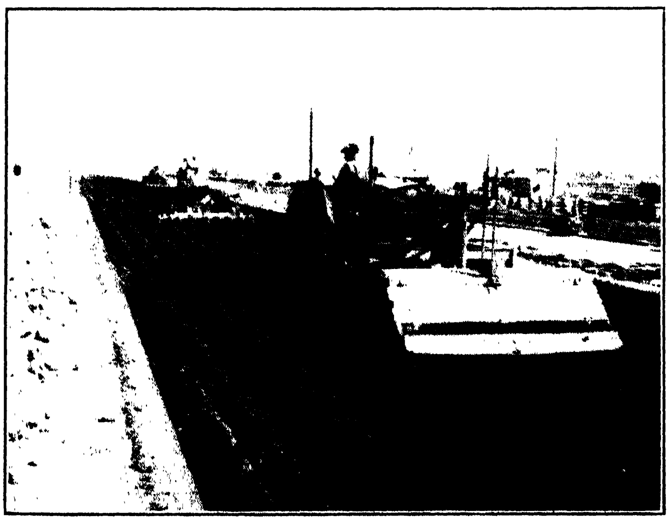
Courtesy Portland Cement Association

FIG. 182.



Courtesy Portland Cement Association

FIG. 183.



Courtesy Portland Cement Association

FIG. 184.

Courtesy Portland Cement Association

FIG. 185.

Courtesy Portland Cement Association

FIG. 186.

roller to give the final compaction. The drags are being operated on the left of Fig. 190 and the roller on the right.

As a final treatment, the surface is given a light treatment of water and rolled with pneumatic rollers as shown in Fig. 191.

Figure 192 shows a completed lane of soil-cement pavement. Note the edge and depth of slab. The steel forms have been removed and they are resting on the pavement. Dampened paper was used in curing.

Soil-cement pavements are subjected to static bearing tests. Figure 193 shows part of the equipment used on such a test. A 30-inch steel bearing plate with Ames dials for recording the deflections caused by the application of the load, shown in Fig. 194, is also used.

154. Flight Strips. *Structural Design.* The following summary gives some of the data used in the design of flight strips.

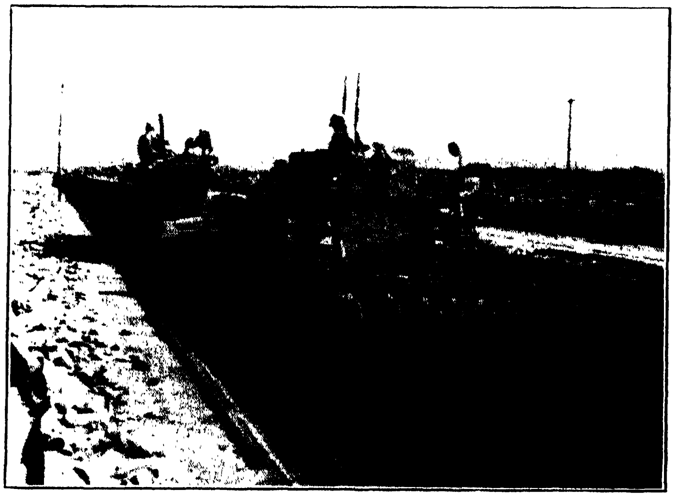
1. Design a subgrade of sufficient and uniform bearing power.
2. Wheel loads of at least 12,500 pounds should be used.
3. Impact may be neglected.
4. Unnecessary to consider fatigue of the concrete as a factor in the structural design.
5. The structural strength depends upon the thickness and support of a pavement. The same theories for thickness may be used as used for airport runways.

155. Drainage for Flight Strips. Drainage is a major problem. It is simpler than the drainage for an airport because only one runway is involved on a flight strip, whereas several intersecting runways are used in an airport. Often the surface water may be provided for by shallow open ditches along the outer edges of the area. When storm drains are needed, the methods used on airport drainage apply.

156. Fencing Flight Strips. Fencing may be an important factor in the layout of a flight strip to prevent cattle from wandering on the landing areas. These fences should be erected at the boundaries of the flight strip.

157. Construction of Flight Strips. The principal construction operations for a flight strip are clearing and grubbing, grading, drainage, and paving. These are all similar to the operation used in the construction of an airport.

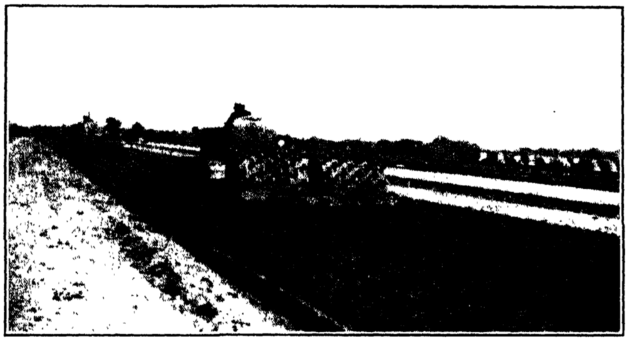
158. Airfield Mats. In 1928 the idea of using open steel mesh grating for airfield runways was fathered by Walter E. Irving, President of Irving Subway Grating Company. At that time he tried, unsuccessfully, to sell various prospective users on this idea, suggesting that the grating could be laid on roofs of buildings or on the ground, or elevated above the ground either slightly or over freight yards. The latter provided the advantage of the midtown location where air and rail travelers and freight could be conveniently transferred to save time and expense of hauling from outlying airports to freight and passenger terminals.



Courtesy Portland Cement Association

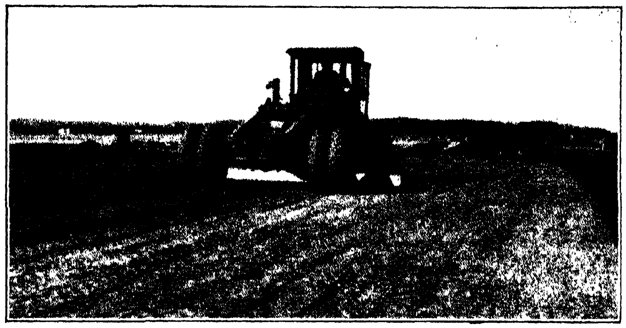
FIG. 187.

The Irving airfield mat consists of units approximately $12\frac{1}{2}$ feet by 2 feet made of alternate straight and reticulate bars of 1-inch by $\frac{3}{16}$-inch strip steel, placed on edge and



Courtesy Portland Cement Association

Fig. 188.



Courtesy Portland Cement Association

Fig. 189.

Courtesy Portland Cement Association

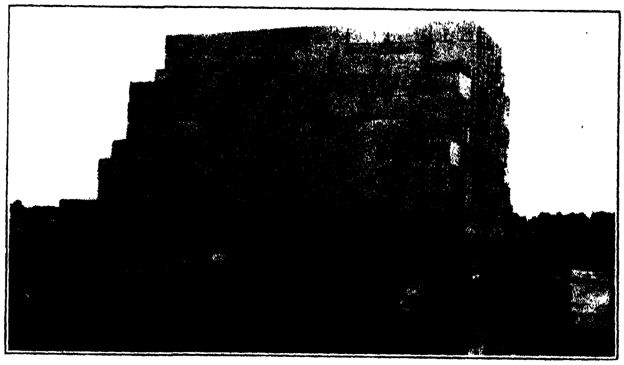
Fig. 190.

Courtesy Portland Cement Association

Fig. 191.

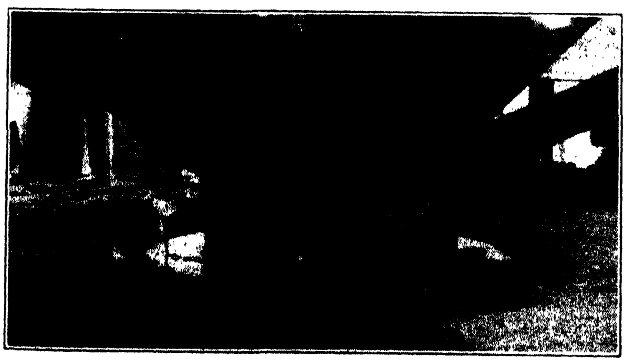
Courtesy Portland Cement Association

Fig. 192.



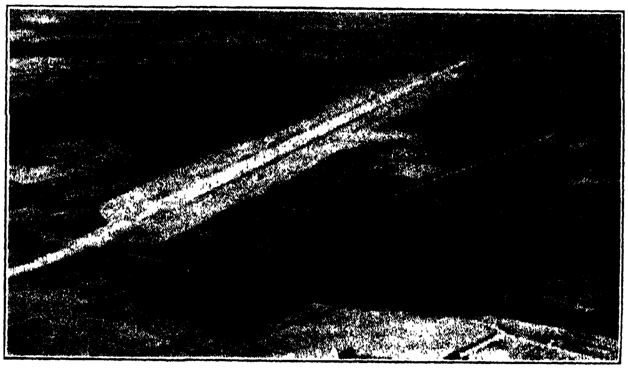
Courtesy Portland Cement Association

Fig. 193.



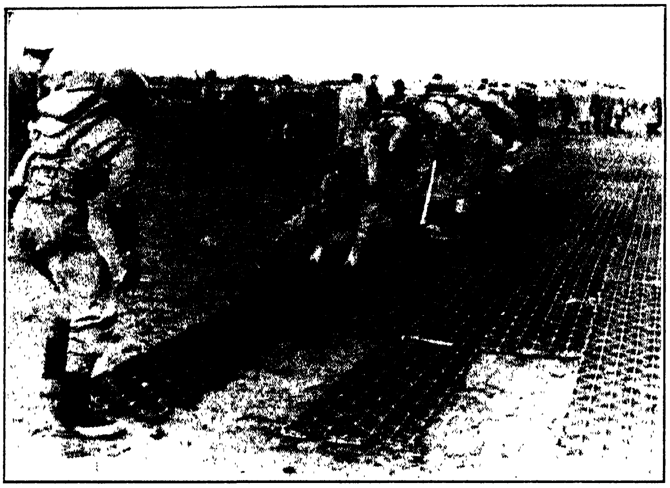
Courtesy Portland Cement Association

Fig. 194.



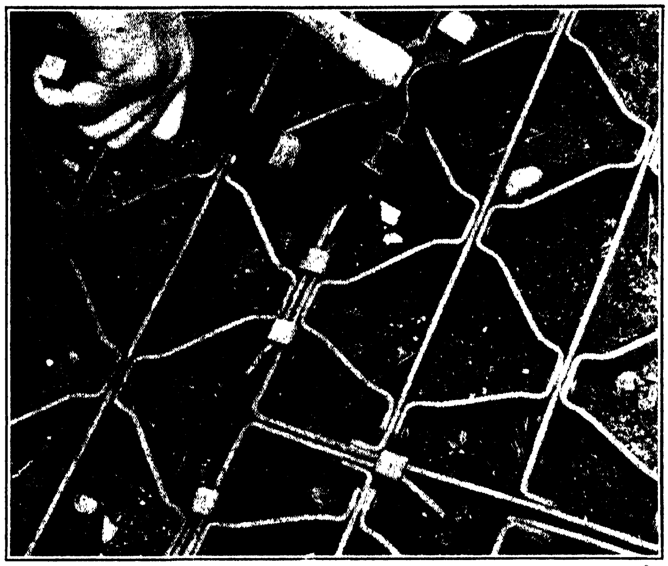
Courtesy Portland Cement Association

Fig. 195. Air view of Army's first concrete flight strip.



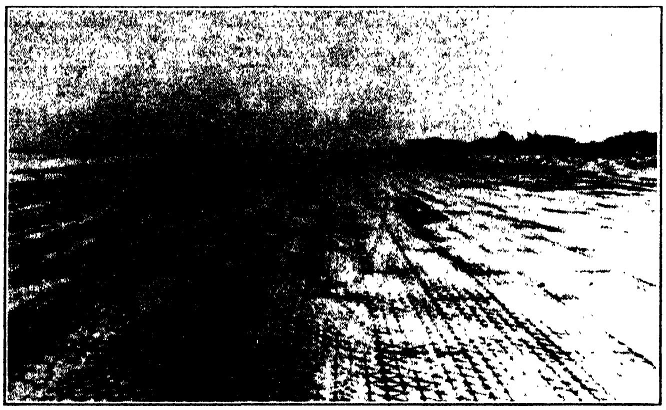
Courtesy Irving Subway Grating Co.

FIG. 196. Soldiers laying a runway of Irving airfield mats.



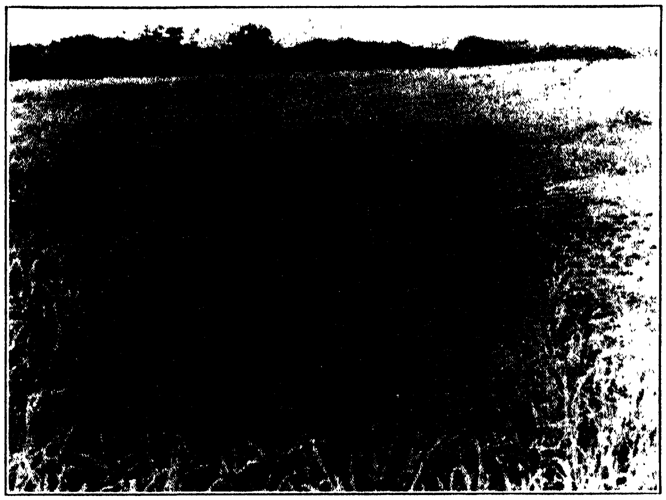
Courtesy Irving Subway Grating Co.

FIG. 197. Method of bending spearheads to bind coupling rings in position.



Courtesy Irving Subway Grating Co.

FIG. 198. Irving airfield mat laid on a beach.



Courtesy Irving Subway Grating Co.

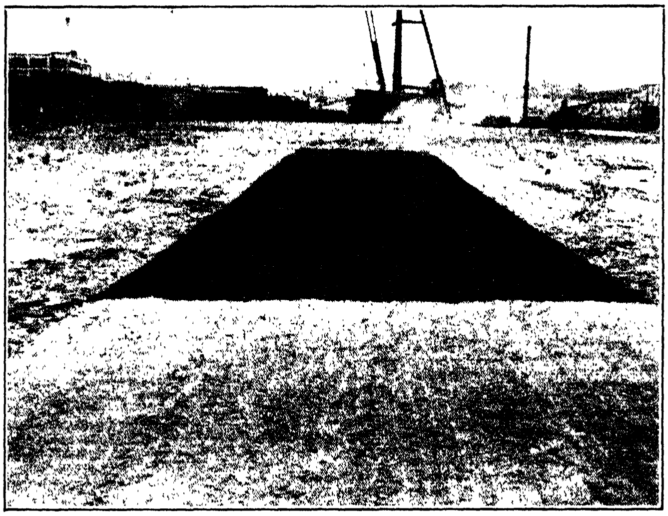
FIG. 199. Irving airfield mat laid on grassy ground.

riveted together at points of contact, the rivets being on 7-inch centers. The straight bars are 3 11/16 inches on centers. Each unit weighs approximately 128 pounds. The outside edges of each unit are equipped with rings and spearheads. When units are laid in position and connected by sliding the rings over spearheads of adjacent units, a runway of any length and width is provided with the seams equally as strong as any other portion. Such runways will accommodate the heaviest bombing planes and are now in use in all theaters of war.

Another type of airfield mat has been developed by Carnegie-Illinois Steel Corporation. It is called U.S.S. Air-Dek and frequently referred to as the "magic carpet." Figures 202 and 203 illustrate this type of landing mat which is made up of a large number of perforated steel sections locked together. A complete air strip may be constructed in a matter of hours; the bearing surface supplied by the flat steel sections increases the bearing power of soft soils.

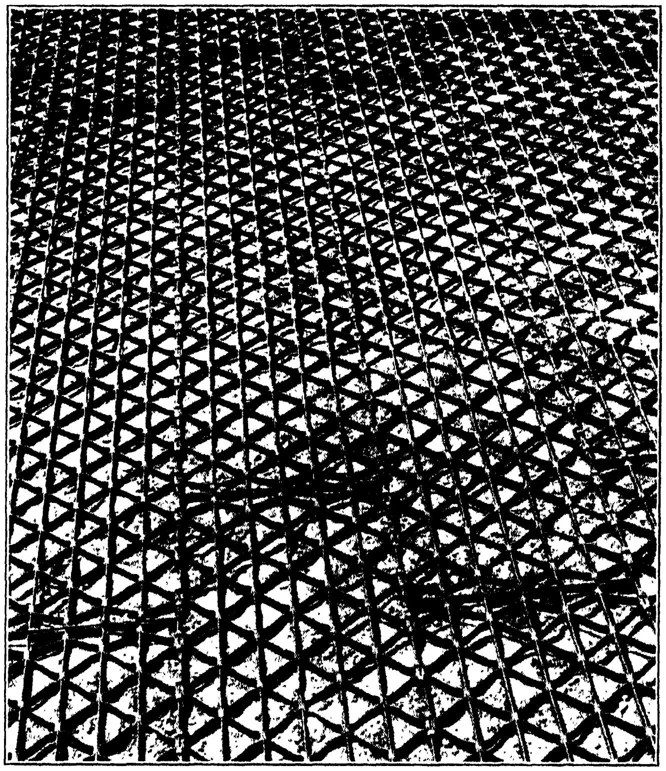
These units may be laid on any reasonably smooth and level ground such as can be found or readily made on a farm or beach; soon thereafter they are less visible from the air than any other type of airfield. Such runways are approximately 90 per cent open and foliage may grow up between the bars to conceal them completely. On sandy beaches, during installation almost enough sand works up within the meshes to camouflage the runways. The addition of a little more camouflages them completely.

After the war, it is quite likely that our country will be dotted with similar airfields because of the simplicity and comparative inexpensiveness of installing this type of runway. When these mats are used for airfield runways of a permanent nature, it is only necessary to fill the meshes with sand and apply a coat of road oil. In tests made with similar material on the Irving Company's proving ground adjacent to its plant, it was found that the sand and oil protected the steel to such an extent that after twelve years of service it was in excellent condition and good for many more years of service.



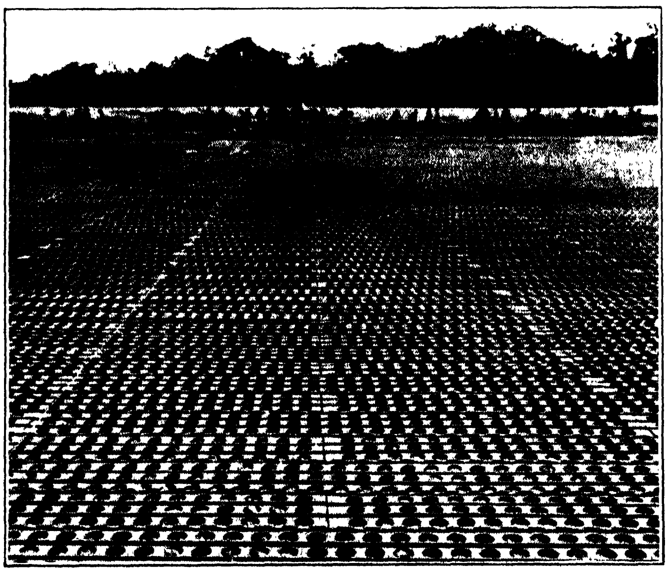
Courtesy Irving Subway Grating Co.

FIG. 200. Strip of Irving airfield mat used as roadway—partly filled with sand.



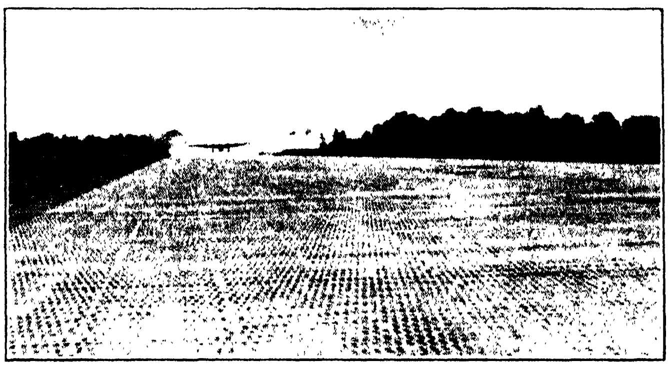
Courtesy Irving Subway Grating Co.

FIG. 201. Part of an Irving airfield mat showing close-up of a number of panels.



Official U. S. Navy Photograph. Courtesy Carnegie-Illinois Steel Corp.

FIG. 202. Seabees constructing an airfield using specially constructed steel mats. These airstrips may be constructed in a very short time, even during battle.



Official U. S. Navy Photograph. Courtesy Carnegie-Illinois Steel Corp

FIG. 203. U.S.S. Air-Dek landing mat. Runway constructed by Seabees at an outlying Pacific base.

Chapter X

Airport Lighting

159. Whether or not an airport is to be used for night flying, it should be provided with lighting equipment to facilitate safe landings at night. Table XXII lists the C.A.A. minimum lighting equipment which should be provided at airports of the various classes.

TABLE XXII

RECOMMENDED STANDARDS FOR AIRPORT LIGHTING

Minimum Recommended Facilities	Class 1	Class 2	Class 3	Class 4 and 5
Airport beacon *	Include	Include	Include	Include
Boundary lights	Include	Include	Include	Include
Range lights	Include	Include	Include	Include
Obstruction lights	Include	Include	Include	Include
Illuminated wind cone	Include	Include	Include	Include
Apron floodlighting	......	Include	Include	Include
Traffic pistol light	....	Include	Include	Include
Landing area floodlighting	....	....	Include †	Include †
Illuminated wind tee	....	....	Include	Include
Ceiling projector	....	....	Include	Include
Runway contact lights	....	....	Include	Include
Approach lights	....	....	Include ‡	Include ‡

Note. All lighting facilities provided in any case should conform to the requirements of the Standard Specifications for the Installation of Airport Lighting Equipment and Materials, issued by the C.A.A.

* Auxiliary beacons, including identification code beacons, should not be set up as a specific requirement for an airport, but will depend on the individual requirements in each case.

† Landing area floodlights are not considered necessary where runway contact lights are installed.

‡ Approach lights should be installed on every runway equipped for instrument landing.

160. Airport Beacon. The airport beacon is a light of the searchlight type which rotates to give a definite periodic flash visible to the pilot from any direction of approach. The light is alternate white and green flashes when viewed from a fixed point. The beacon should be placed at least 20 feet above any building and as near to the landing area as possible and at a point where it will be a minimum obstruction to flying. The beacon should be placed on a tower which is structurally designed.

If the beacon is placed more than $1\frac{1}{4}$ miles from the field boundary, an auxiliary green code beacon should be installed on the airport.

161. Boundary Lights. The entire available landing area of the airport must be outlined with white lights spaced 300 feet apart and with one light placed at each corner or angle in the boundary. These lights give the pilot a definite picture of the landing area. The lights are usually mounted on sheet metal cones with the light between 30 and 36 inches above the ground. Flush-type lights must be used where the boundary lights are within the 1000-foot wide approach zones of an instrument landing runway, or where they are used to mark the edge of a hangar or loading apron, or where the marked boundary crosses a taxi strip leading off the field.

162. Range Lights. Green lights are placed in the line of boundary lights to mark the ends of runways or favorable landing directions. These range lights should be set in groups, with the same number of lights at the ends of the same runway and different numbers of lights for different runways, the largest number marking the most important runway. The lights are placed about 50 feet apart and must not extend beyond the projected edges of the runway and the groups marking different runways must be separated by at least one white light. If contact lights are used to mark the runways, four range lights are used to mark each runway. The same types of lights are used for range lights as for boundary lights—either the cone type or flush type. All groups of range lights should be perpendicular to the axis of the runway.

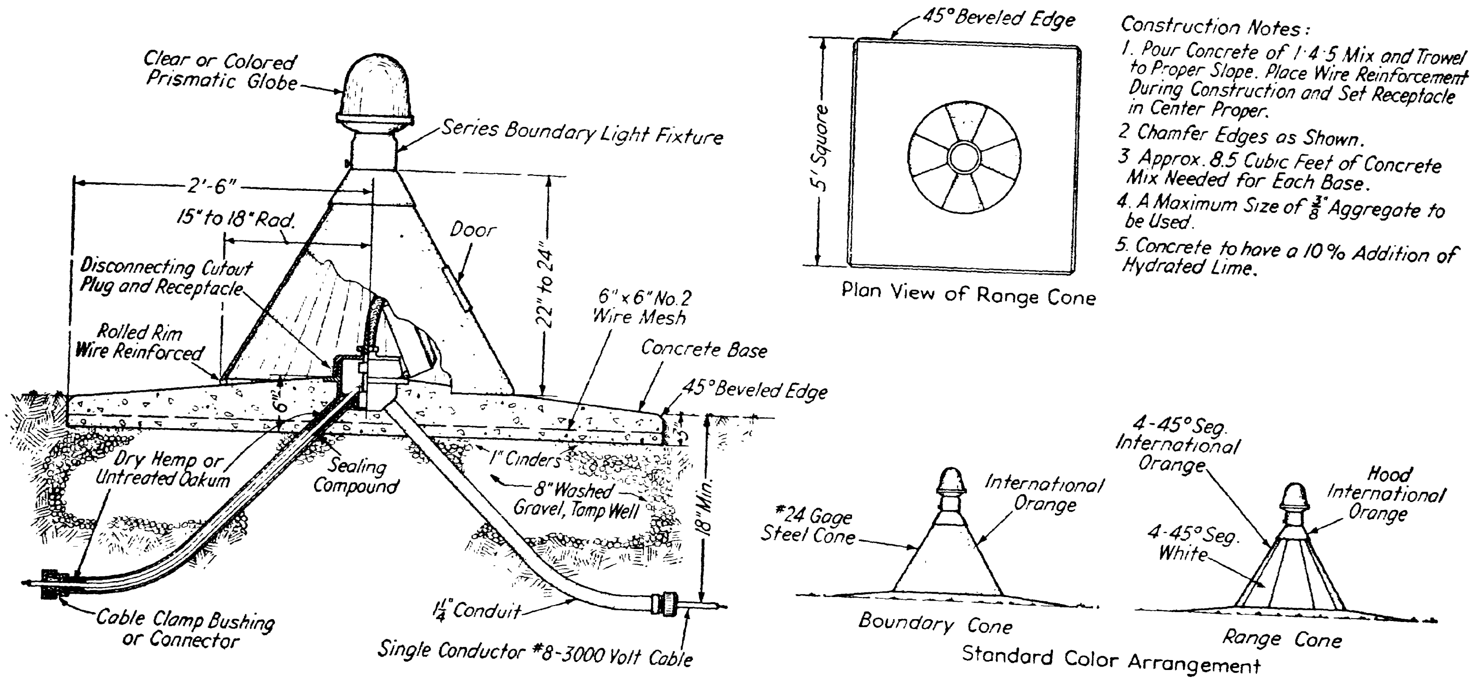
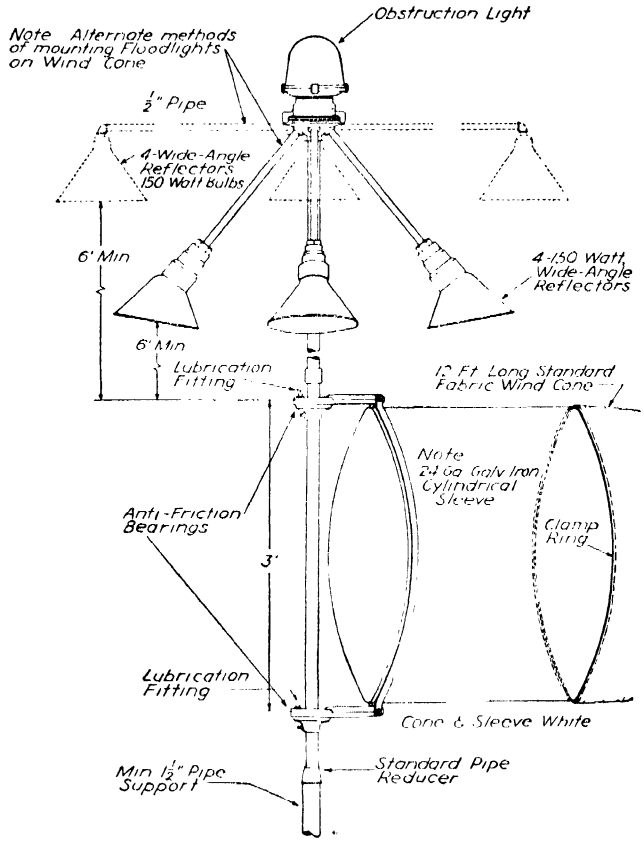


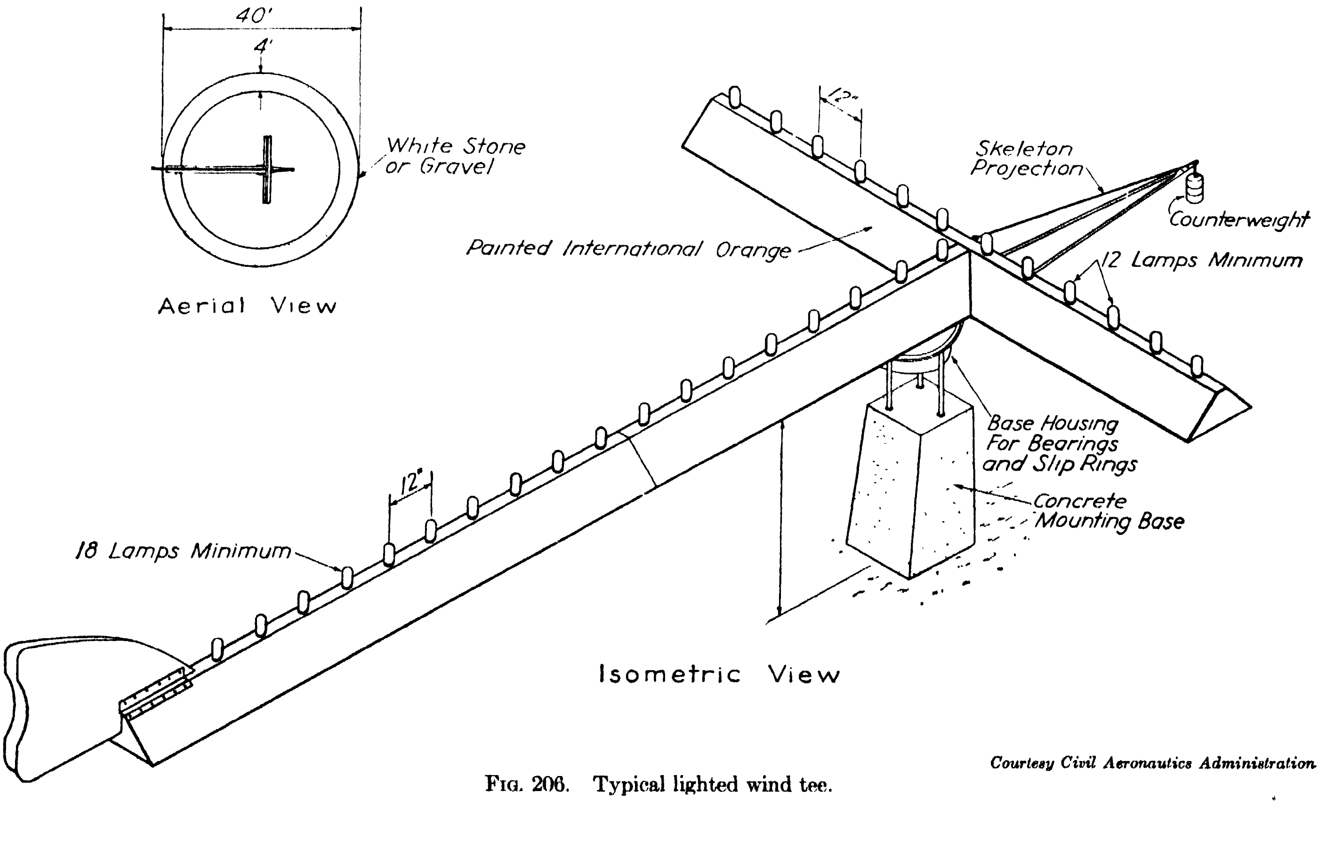
FIG. 204. Boundary and range cone installed with concrete base.

Courtesy Civil Aeronautics Administration



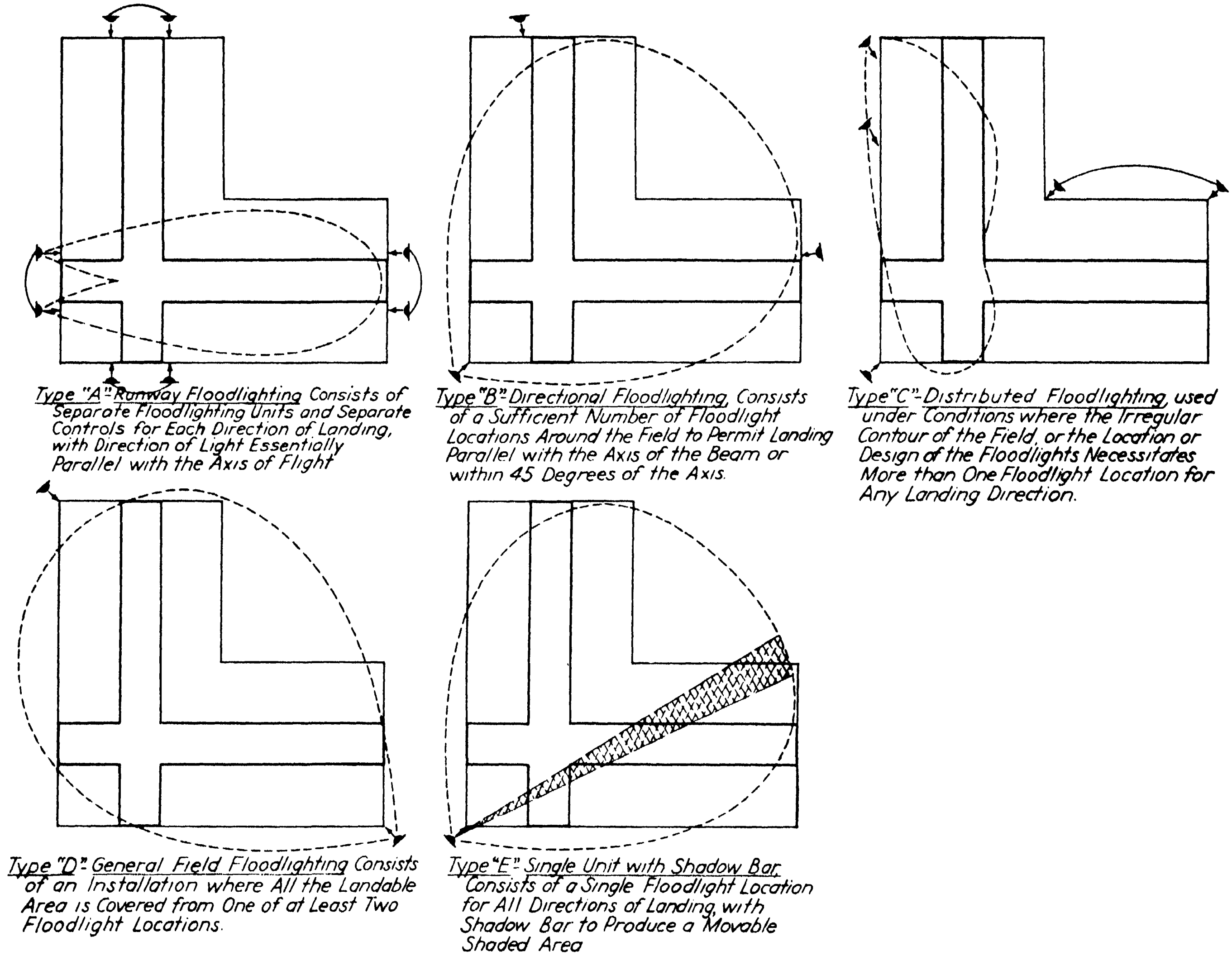
Courtesy Civil Aeronautics Administration

FIG. 205. Typical externally illuminated wind cone.



Courtesy Civil Aeronautics Administration

FIG. 206. Typical lighted wind tee.



Courtesy Civil Aeronautics Administration

FIG. 207. Five general types of airport lighting.

163. Obstruction Lights. All obstructions that extend above a slope of one-in-twenty taken through any point at the edge of the boundary area or above one-in-thirty at the approaches of the runways should be removed if possible. If this is not possible they must be marked with obstruction lights which are red and must be visible from 15 degrees below the horizontal to the zenith. Obstruction lights are placed on all airport obstructions such as buildings, floodlights, wind cone supports, and beacon towers; and also on neighboring obstructions such as pole lines, trees, radio towers, buildings, and so forth.

164. Illuminated Wind Cone. At least one illuminated wind cone is necessary in each airport to give the true direction of the wind at all times. It should be placed at a point where it will not be affected by eddies caused by buildings or other features and where it may be seen from all directions of approach.

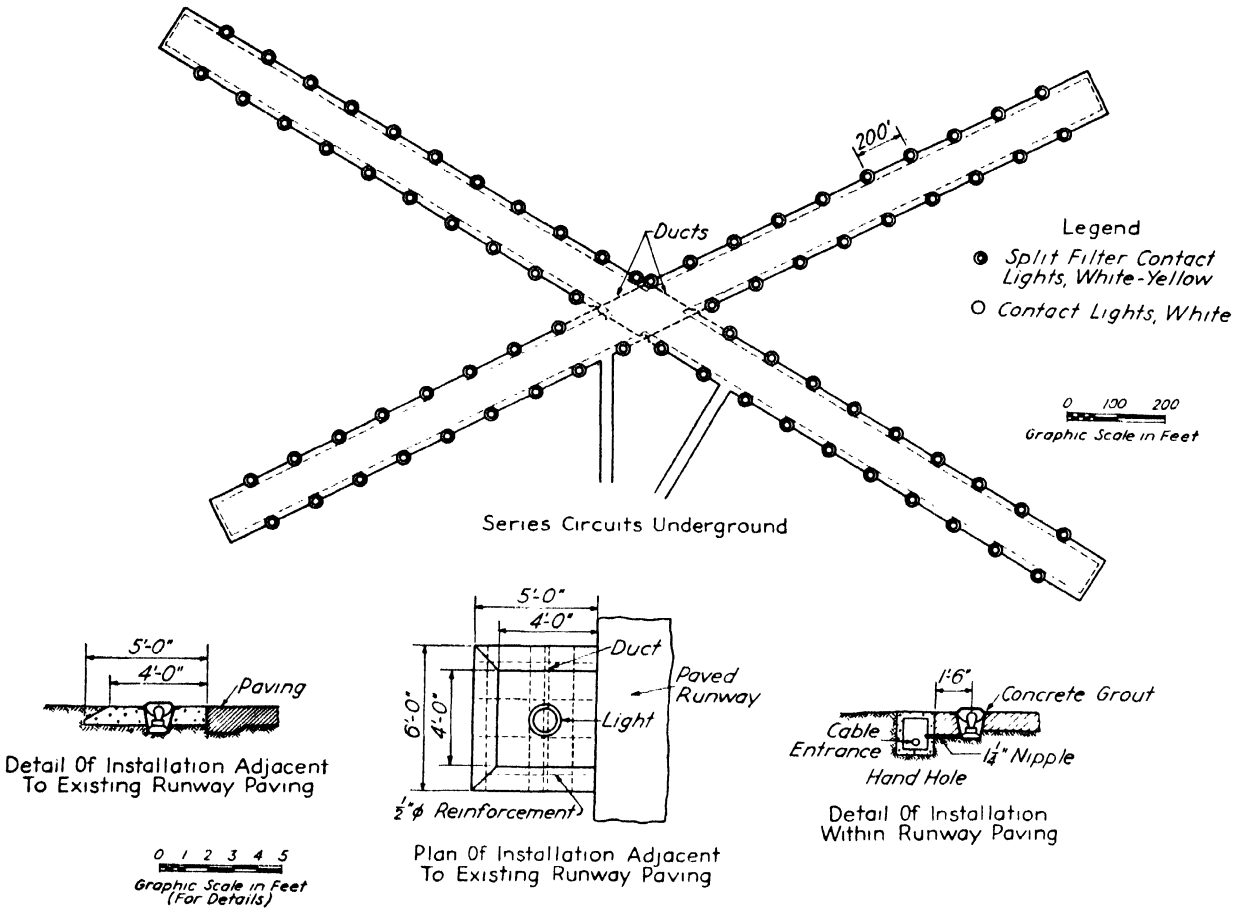
A wind tee or tetrahedron wind indicator serves the same function as the lighted wind cone.

165. Apron Floodlighting. The aprons and loading areas should be floodlighted for convenience in servicing and loading. Usually these lights are mounted on hangars or other buildings.

166. Landing Area Floodlights. Five general methods of floodlighting the landing area are used. These are shown in Fig. 207.

167. Ceiling Projector. In order to determine the height of the lowest clouds at night, a light which projects a narrow beam of light in a vertical direction is used in conjunction with a clinometer. This light is usually placed 1000 feet from the point at which the clinometer readings are to be made; the height of the clouds at night above the airport may then be found by triangulation.

168. Runway Contact Lights. Contact lights, consisting of white lights mounted nearly flush with the ground and spaced 200 feet apart along both edges of the runways, are used at the larger airports to assist pilots in making landings and take-offs. These contact lights are controlled



Courtesy Civil Aeronautics Administration

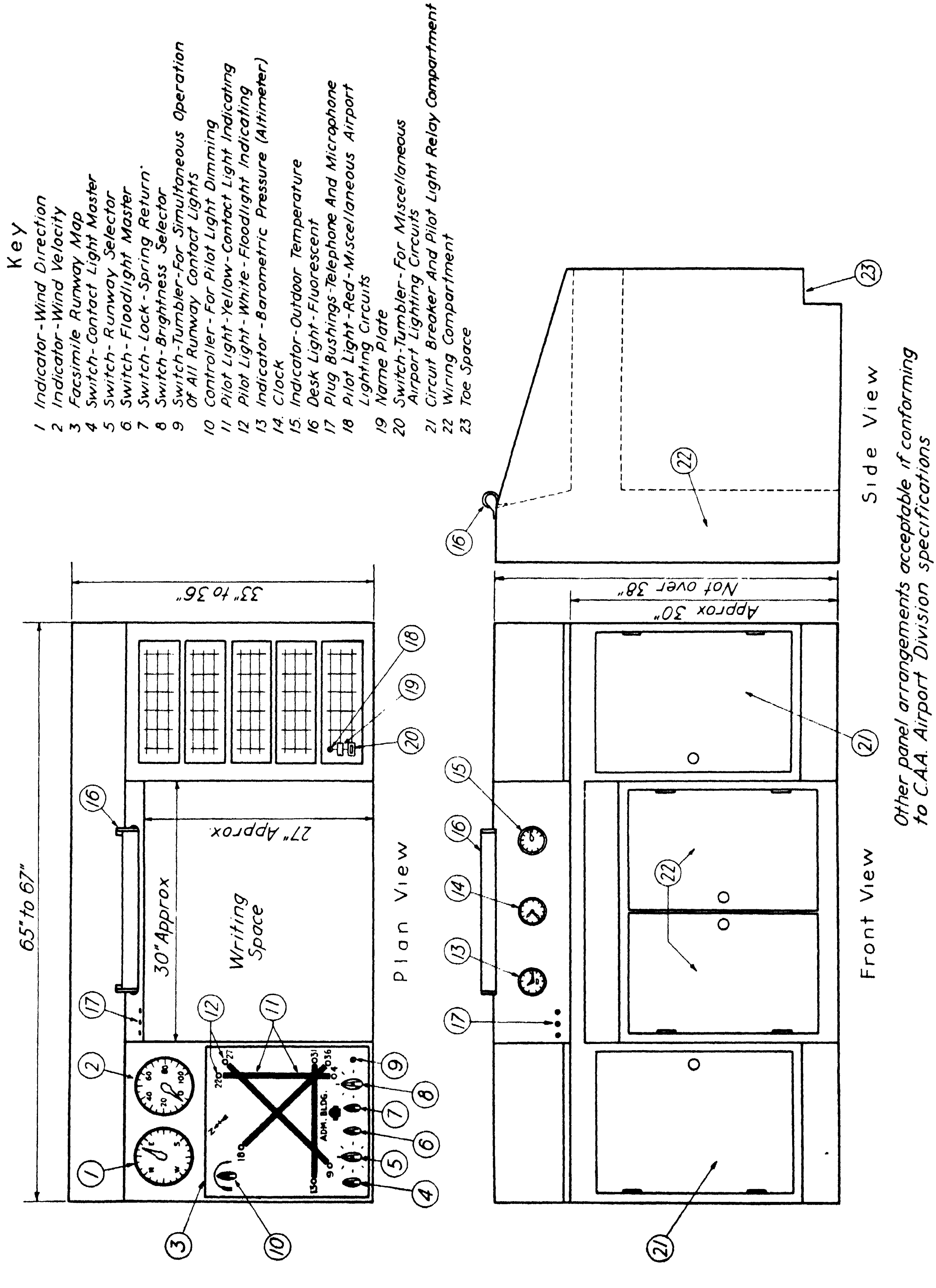
FIG. 208. Typical contact light installation.

separately so that only one runway is illuminated at one time. This will indicate to the incoming pilot which runway he should land on; it also assists a pilot, when taking off, to line up his ship properly with the axis of the runway.

Contact lights are white except that along the final 1500 feet of a runway split filters are used on the lights so that these lights show yellow when viewed from the far end of the runway and white when viewed the opposite end. On landing or taking off the yellow lights warn the pilot that he has only 1500 feet of runway left. Contact lights are very effective in poor weather when floodlighting is very unsatisfactory.

169. Control Desks and Control Panels. The airport control desk or panel controls the beacons, boundary lights, contact lights, runway floodlights, obstruction lights, and other field lighting. Control panels are normally used at airports that do not have control towers, or when the tower is not completely glass enclosed. Control panels are usually installed on a wall of the manager's office or of the control room. The panel should be so located that an operator may watch the landing area through a window while operating the control switches.

Control desks are used at large airports having control towers enclosed in glass where an operator is on duty at all times. The control desk is installed at the front of the control tower so that the operator faces the field and has an unobstructed view of the landing area.



Courtesy Civil Aeronautics Administration

FIG. 209. Typical airport control desk.

Chapter XI

Airport Buildings

170. General. The complete airport plan will not neglect the architecture of the airport buildings which are to serve the passengers and planes. As in railroad termini, passengers will be impressed by the facilities and appearance of the airports which are provided by the different cities. The first impression of any community is usually of lasting effect and is worthy of considerable thought when the service buildings at an air terminal are being designed.

The modern airport must provide all the services and accommodations that are found at a railway terminal. In addition, the nature of this means of transportation requires that many facilities, which in rail transportation are located some distance from the terminal, be located on the airport itself.

Let us consider the arrival of a plane carrying freight, mail, passengers, or a combination of all three. The approach of this plane is controlled from the traffic control room by radio. To avoid collisions in the air the vertical and horizontal position must be designated for each approaching plane. The movement along the runways and taxiways after the plane has landed is controlled from this same point. Upon the arrival of the plane at the platform, the contents must be unloaded and passed through the terminal to the surface transportation facilities used to continue it on its way. Separate channels for each class of load should be provided to facilitate this transfer.

Upon unloading, the plane must be moved to a point where it can be thoroughly serviced before continuing on its flight. This servicing must include mechanical inspection, refueling, cleaning, and restocking with food and supplies for passengers and crew. Accommodations for the crew members must be provided in order that they may relax or be relieved by a new crew before the flight is continued.

When the plane is prepared for departure it is returned to the loading platform where passengers, mail, and freight are loaded. It then proceeds to the proper runway and again takes off under orders from the control tower.

The grouping of buildings and the facilities offered by each building should be such as to coordinate efficiently all the functions of the terminus in order that the meeting of air and land transportation lines can be accomplished with a minimum of wasted time and effort.

171. Administration or Terminal Buildings. The administration or terminal buildings do not present any unusual structural problems; the layout and plan follow the same general principles as those of railroad terminals. This building or buildings provide for the necessary services, such as offices of management, ticket office, restaurant, etc. The amount and kind of traffic will determine what facilities must be provided but the following must be considered: airport manager's office, U. S. Weather Bureau, communications office, ticket office, telegraph office, waiting rooms, restaurant, baggage, mail, and express rooms, public telephone booths, check room, concession stands. If an airport traffic control tower is required by traffic handled it is usually located on the roof of the administration building. The plan for a typical small-sized airport terminal building is shown in Fig. 210 and plans for a typical administration building for major airport are shown in Figs. 211 to 216.

In planning any of the buildings for an airport provision for expansion is very important; much money may be wasted in construction if provision is not made for possible expansion. Hangars can be duplicated to handle additional traffic, but there should be only one passenger terminal building. Provision should be made so that interior partitions may be moved for a reallotment of space inside the building.

172. Hangars. The hangars to be provided at an airport must include both service and storage hangars. The service hangars must be provided with machine shops, stores of spare parts and provisions, and ample space to work upon the planes in all kinds of weather. The present tendency is to provide the open floor space in the center of the building with the necessary shops, offices, etc., in rooms along three sides of the hangar building. A typical set of plans for a hangar is shown in Figs. 217, 218, and 219.

Modern equipment permits installation of large overhead or horizontal doors along the long dimension of the hangar. This enables a greater number of planes to be stored in any building without the interference that accompanies the storage when the narrow face of the building is used for the hangar opening, and provides for expansion of the hangars. The hangar openings should have a clear height of at least 26 feet and provision should be made to increase this to accommodate larger planes of the future. This can be accomplished by maintaining a greater clearance height for all structural members such as trusses and girders so that the door openings may be increased without major structural changes throughout the building.

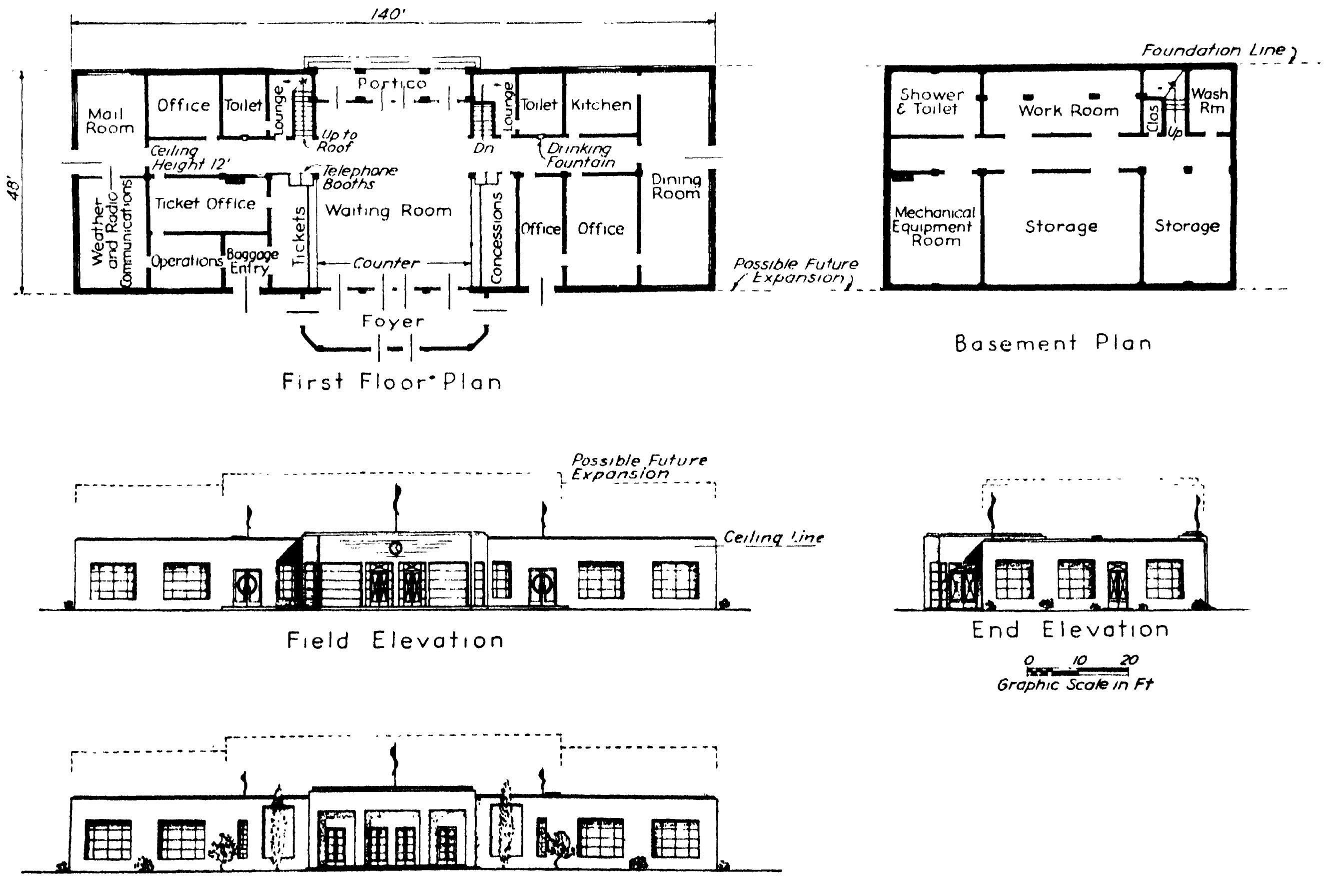
The nature of the work required in servicing planes demands that excellent light and heat be provided at all points within the hangar. Large expanses of glass are used in the walls and ceilings.

The large glass and door openings make difficult the adequate heating of a large hangar in severe weather. The most successful solution of this problem has been the use of separate heating units mounted so that the flow of warm air may be directed at the point where it is most needed.

The number of hangars for servicing and for storage will vary with the size and traffic. Because of the hazard of fire the use of extremely large hangars is not recommended unless they are well protected by fire walls within to prevent the spread of fire from one end of the hangars to the other.

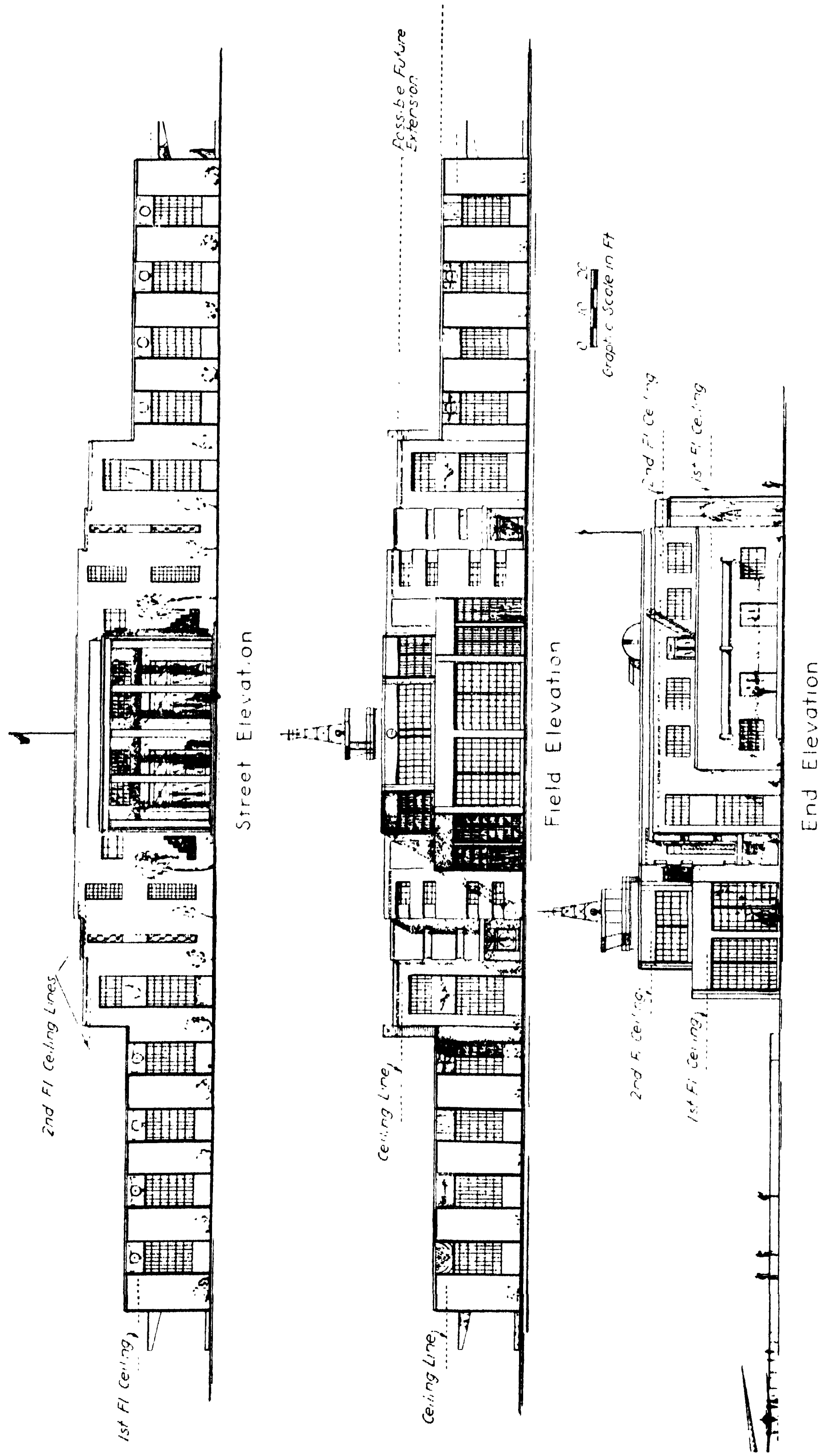
The grouping of the airport buildings should be so coordinated that each will function efficiently under existing traffic conditions and allow for future expansion without requiring expensive changes in the existing structures. The military airport will present a greater problem in planning the grouping of buildings because of the more extensive services required. A modern air base will include, in addition to the administrative building and hangars, buildings to serve as barracks, hospitals, ammunition storage space, general storage space, and other areas necessary for a complete military base.

These buildings should be so grouped that each may serve its designated function as a separate unit as well as contribute the most to the operation of the entire post as an efficient installation. A very careful study of the marginal areas adjacent to the landing area must be made in order that the buildings may be located so that they will not limit further expansion of the runway system.



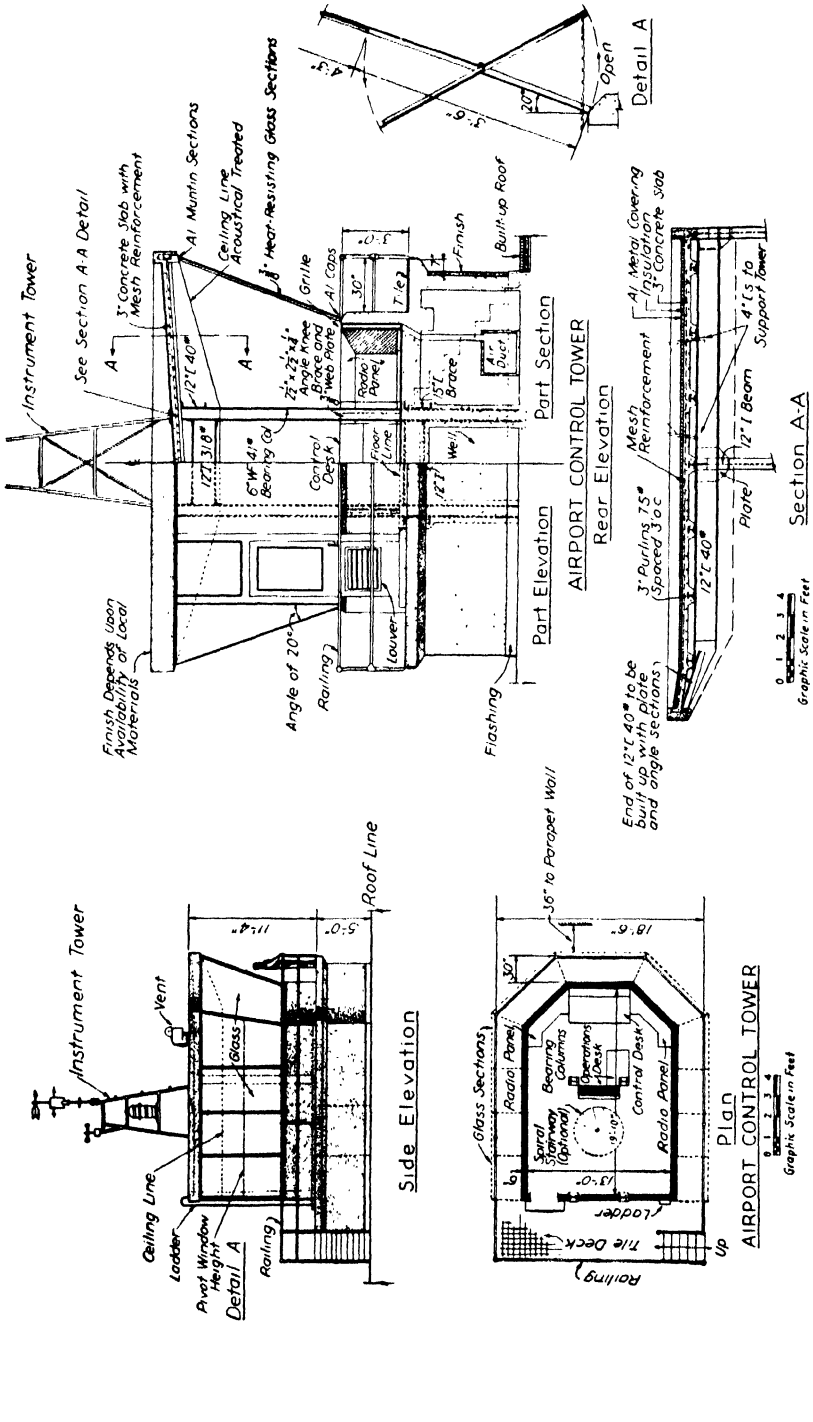
Courtesy Civil Aeronautics Administration

FIG. 210. Typical small-sized airport terminal building.



Courtesy Civil Aeronautics Administration

FIG. 211. Typical administration building for a major airport.



Courtesy Civil Aeronautics Administration

FIG. 212. Airport traffic control tower.

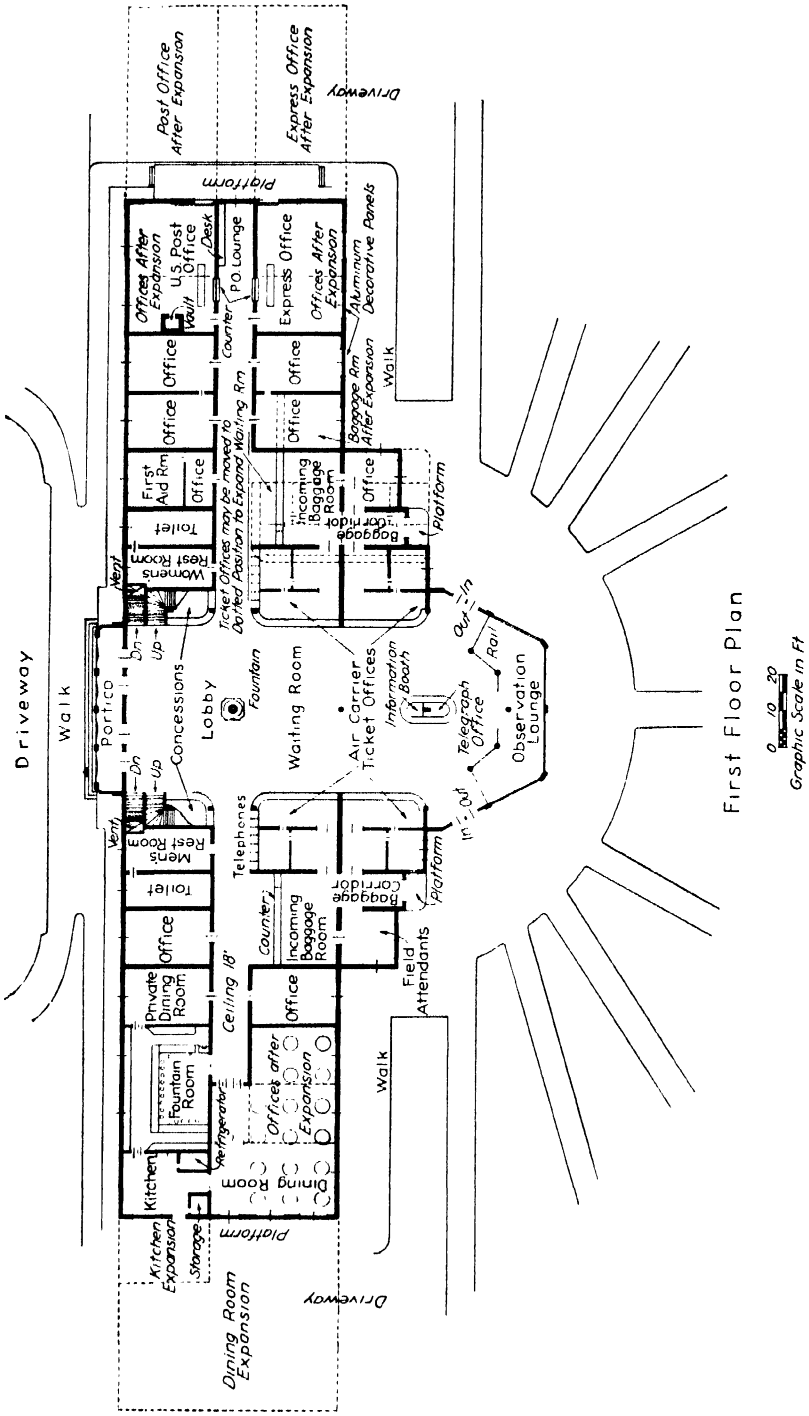
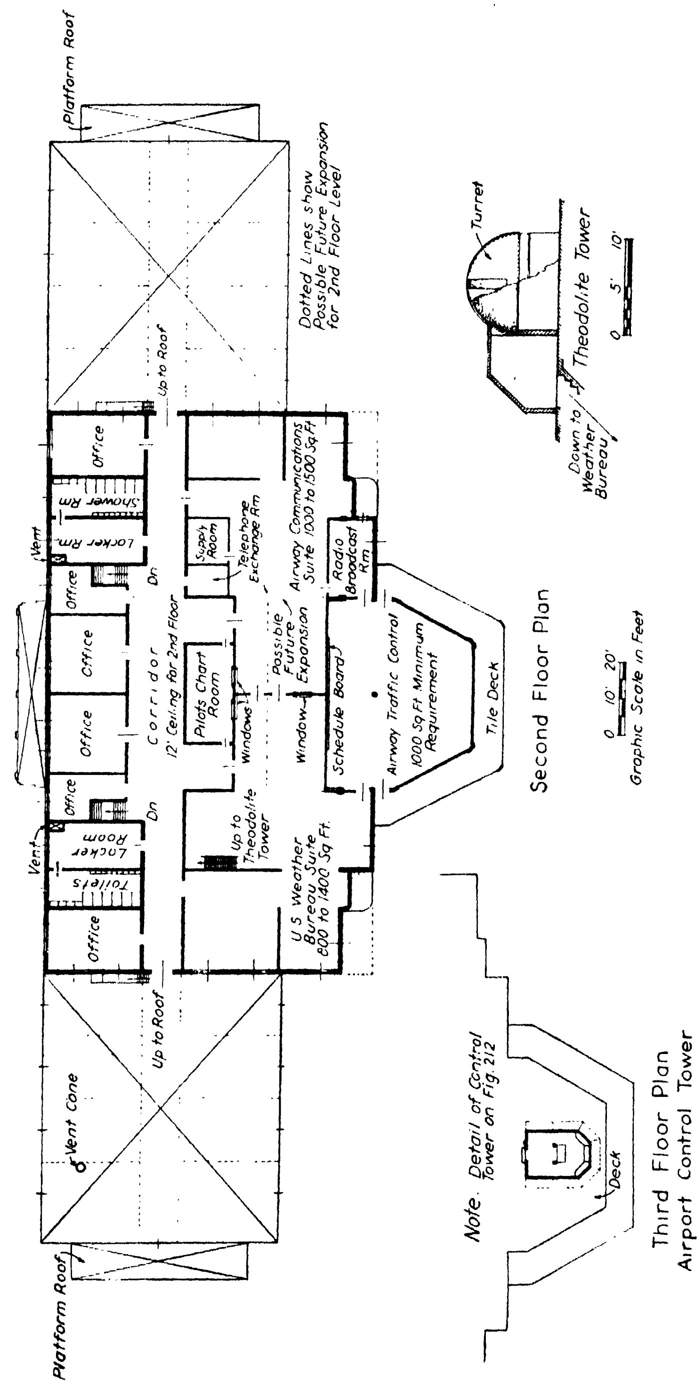


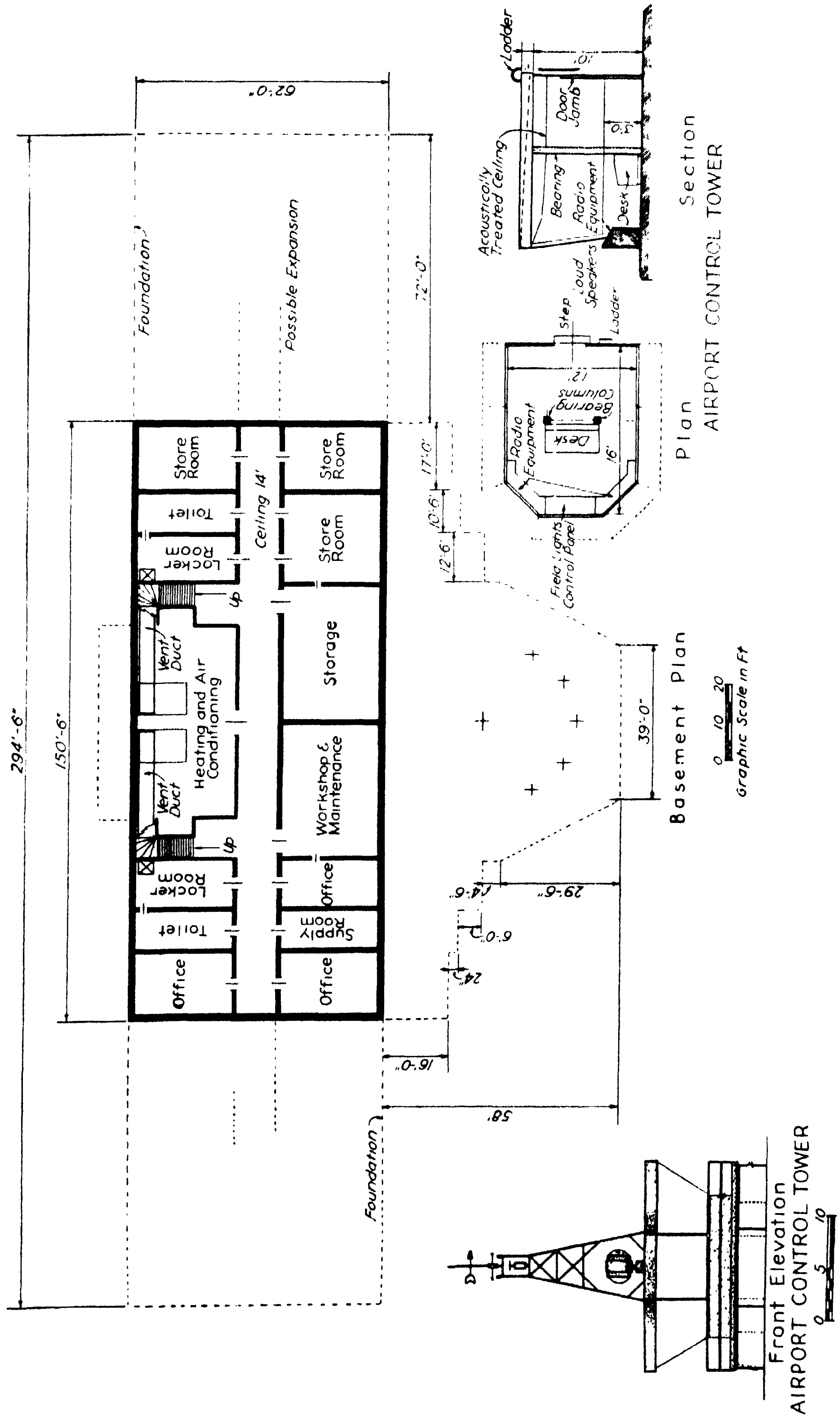
FIG. 213. Typical administration building, first floor plan, major airport.

Courtesy Civil Aeronautics Administration



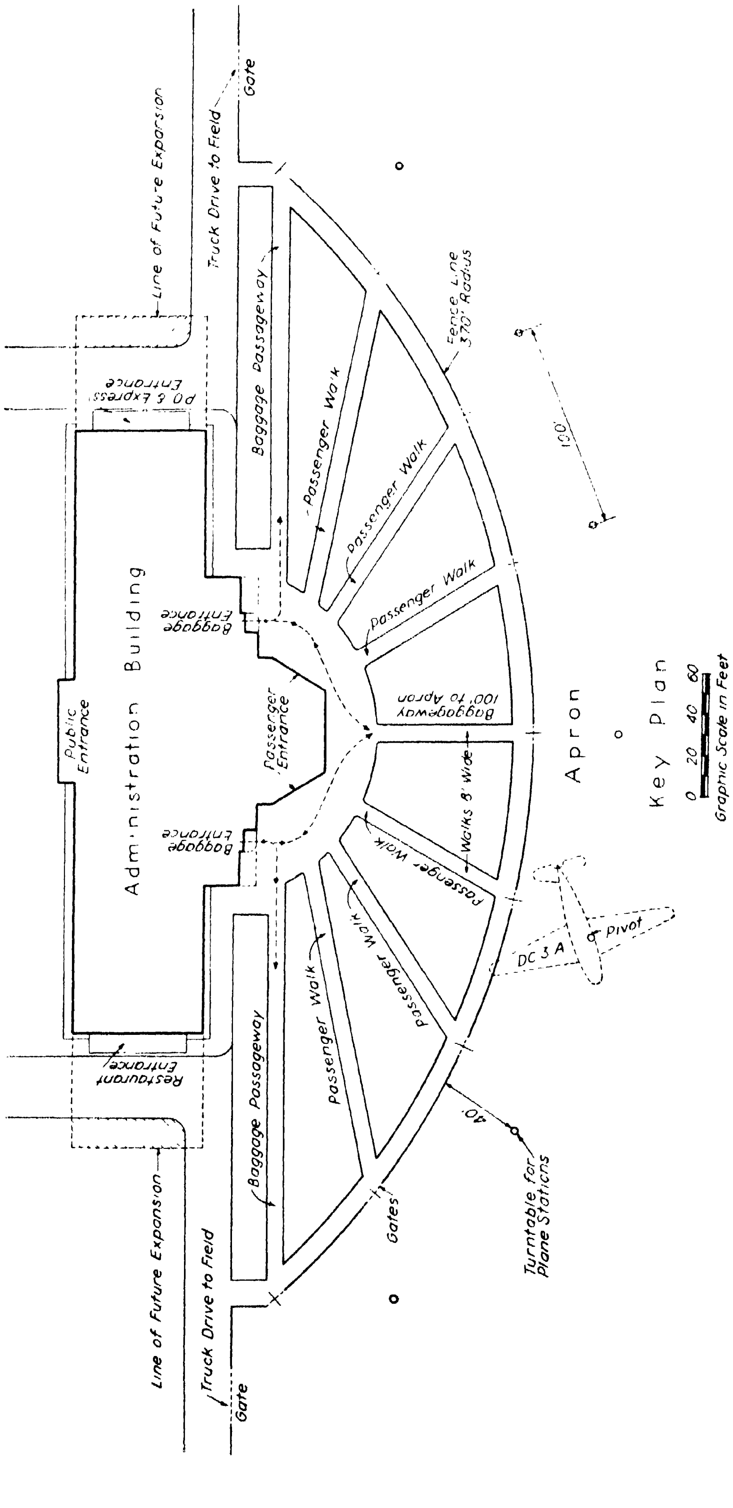
Courtesy Civil Aeronautics Administration

FIG. 214. Typical administration building, second floor plan, major airport.



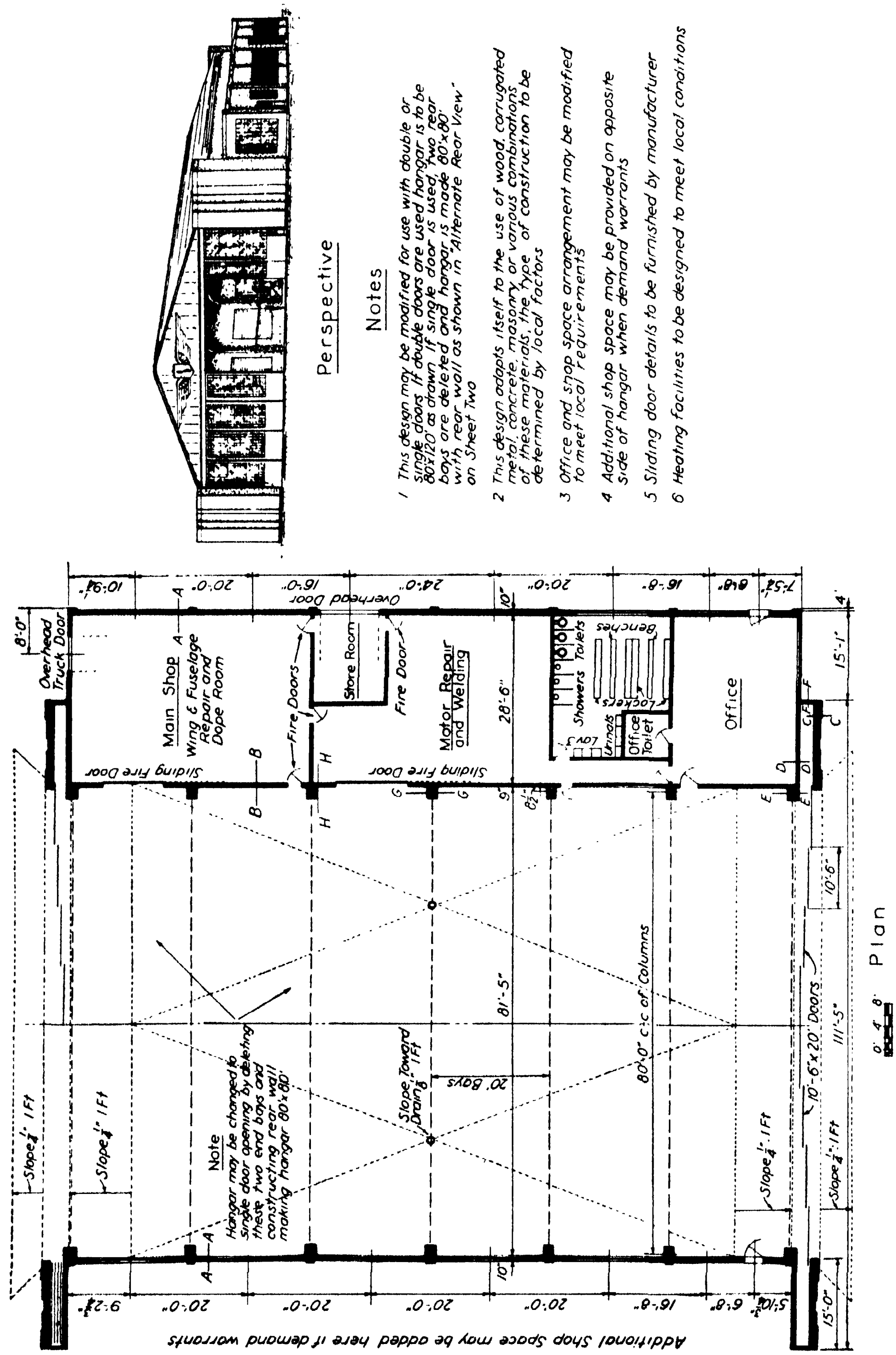
Courtesy Civil Aeronautics Administration

FIG. 215. Typical administration building, basement plan, major airport.



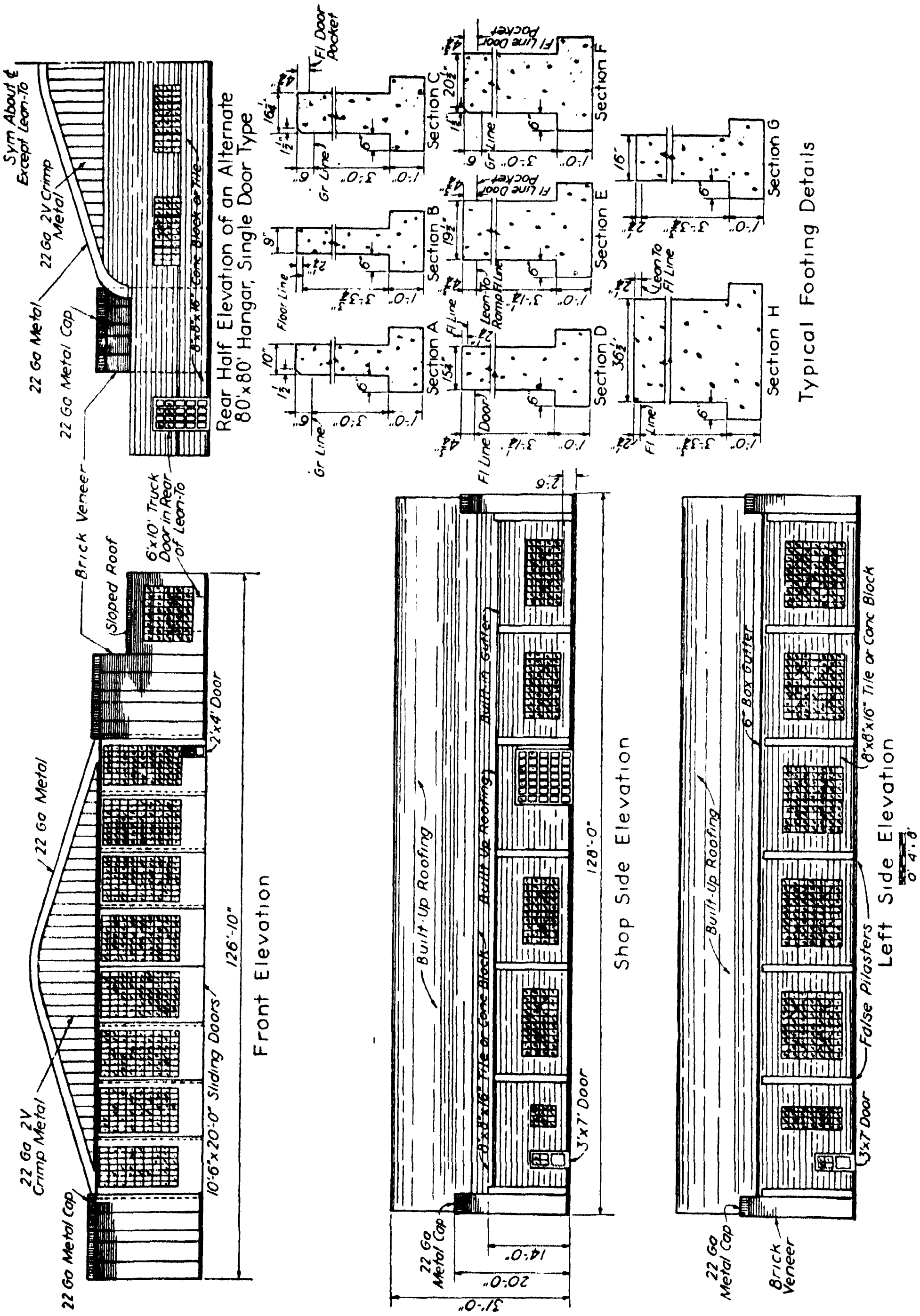
Courtesy Civil Aeronautics Administration

FIG. 216. Typical administration building, key plan, major airport.



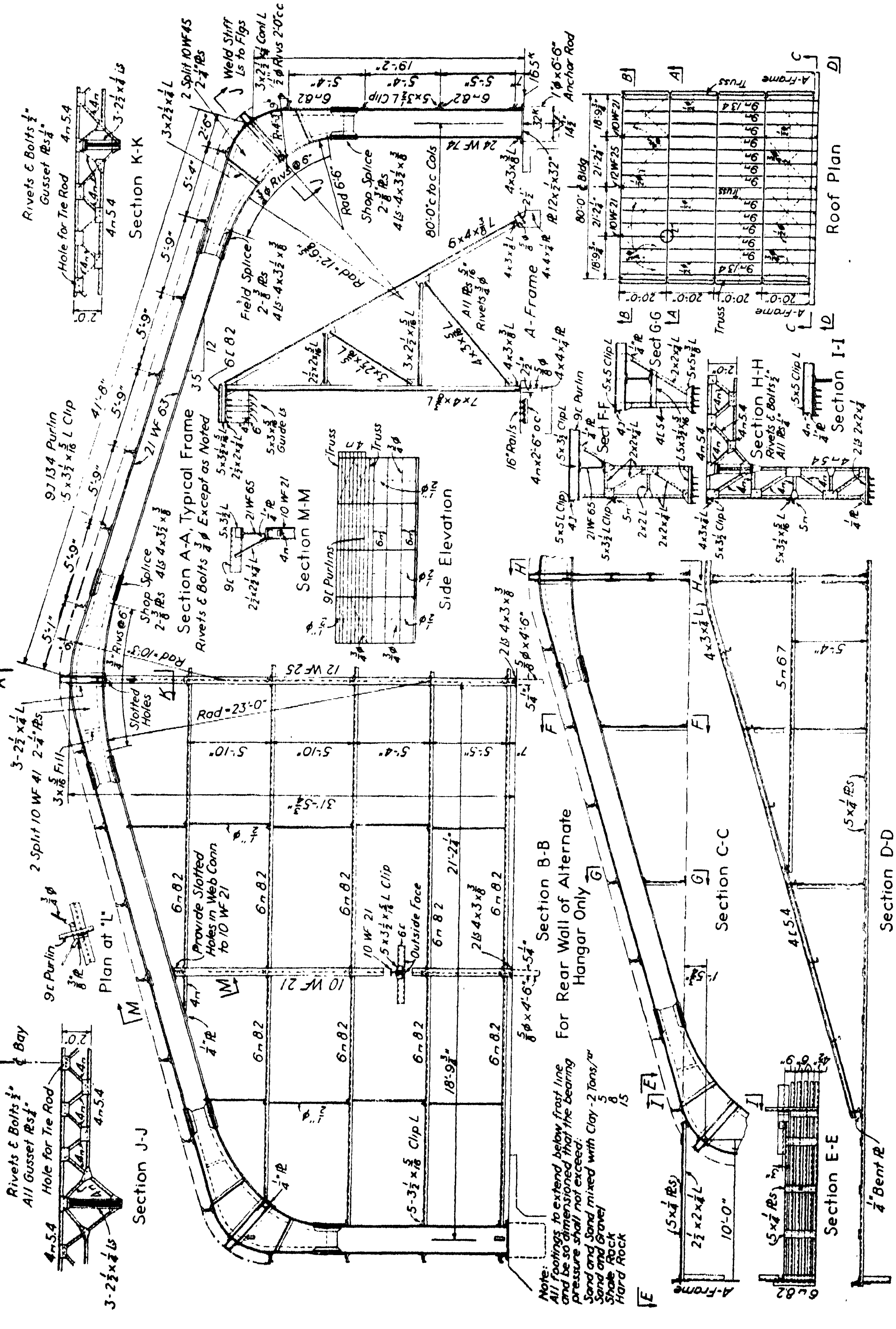
Courtesy Civil Aeronautics Administration

FIG. 217. Rigid steel frame hangar, 80 × 120 feet.



Courtesy Civil Aeronautics Administration

FIG. 218. Rigid steel frame hangar, 80 × 120 feet.



Courtesy Civil Aeronautics Administration

FIG. 219. Rigid steel frame hangar, 80 × 120 feet.

Chapter XII

Construction and Maintenance Equipment

173. General. Airport construction in a way is simpler than highway construction as it is not necessary to maintain traffic as it usually is on highway construction. The largest type of equipment may be employed with plenty of room for moving it about. Large amounts of cut-and-fill are involved and it is only through the use of modern equipment that the unit cost is held down.

174. Hydraulic Fills. Many airports are built at or close to sea level and many have been designed to use areas which were originally covered by shallow water. Such a design is shown in Fig. 220.

The fill for such an airport may be made by pumping the fill in place by hydraulic methods. Figures 221 and 222 show in place the pipe through which the fill is being discharged.

Hydraulic methods are economical and rapid for making fills near large bodies of water.

175. Grading. It is safe to say that most airports are located in places where hydraulic fills cannot be used. Other methods must then be used to excavate and transport the earth from one place to another. Since large amounts are involved, it requires some study by the engineer to select the methods and equipment best suited to the particular job.

It has been said that two developments have been largely responsible for modern economic construction of

Fig. 220. Area to be filled by hydraulic methods.

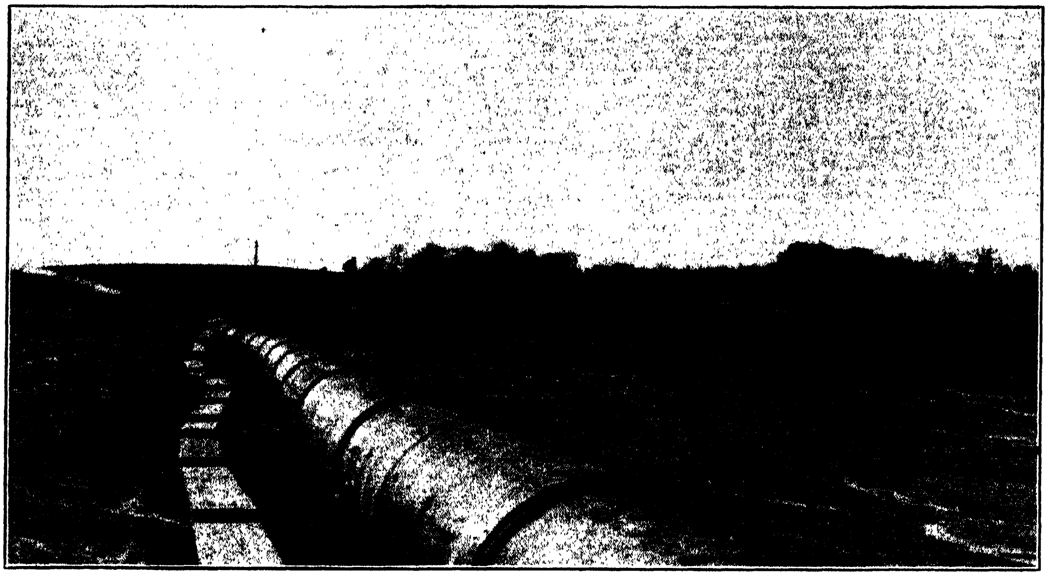


FIG. 221. Pipe line discharging hydraulic fill along runway location.

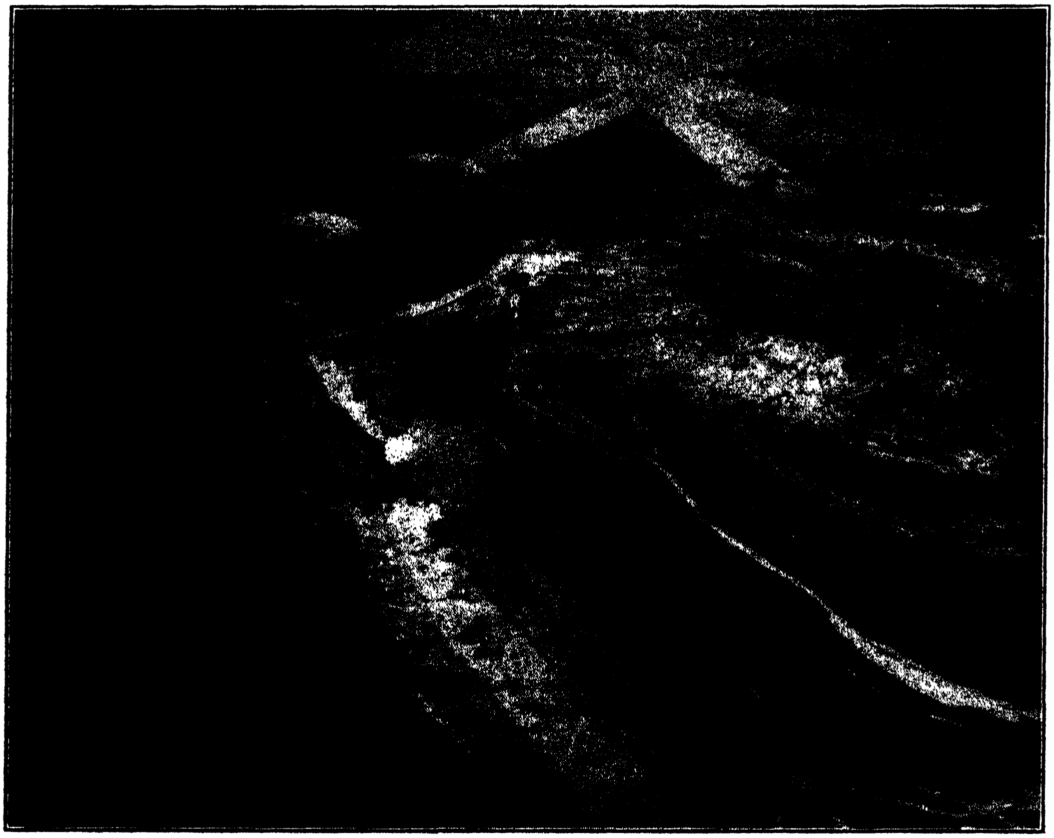
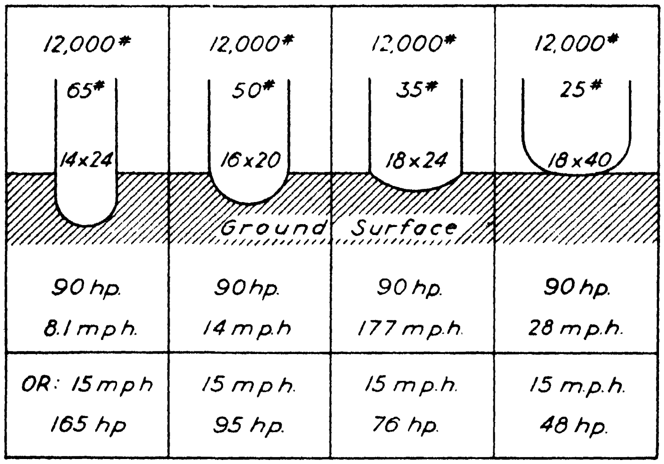


FIG. 222. Pipe line used for hydraulic fill.

highways and airports. They are the Diesel engine and alloy steels. The Diesel furnishes economical power. The alloy steels are used in cutting, scraping, and hauling equipment to resist the tremendous strain placed upon it by the huge tractive force of the Diesel-driven tractors.

It will be noticed that most of the trailing equipment is tired with large tires. At first thought this may seem extravagant as these large tires are very expensive. Reference to Fig. 223 will show the effect of various tire sizes



Courtesy LeTourneau

FIG. 223. Effect of different tire sizes.

and pressures on speed and power. This chart is the result of studies made by the engineers of R. G. LeTourneau, Inc.

Let us assume that the necessary surveys have been made, the plans have been drawn, and the necessary staking is completed. The type of earth to be excavated and moved as well as the length of haul will give the engineer some valuable information to use in the selection of equipment.

If rock excavation is involved, it may be necessary to resort to blasting, with power shovels for loading dump wagons hauled by tractors.

Figure 224 illustrates a shovel loading an Athey side-dump wagon which is usually hauled in tandem by a tractor.

Figure 225 illustrates a tandem hook-up of side-dump wagons in transit; the economy of operation of such a unit is indicated by the fact that the fuel consumption was 30 gallons per 8-hour day at a cost of 6½ cents per gallon.

Figure 226 illustrates the method of dumping material in the fills; the operation of these dump wagons is controlled from the power unit and by the driver of the tractor.

Many airports are built on sites which do not involve rock and deep cuts. The operation in such cases involves loosing the material so that it can be picked up by some scraping equipment.

Figure 227 shows a rooter at work on airport construction. This equipment is extremely powerful and it is economical to operate.

If short hauls are involved, a revolving scraper may be used. This equipment is illustrated in Fig. 228. However, the haul is likely to be long on airport construction and larger equipment must be used. Figure 229 illustrates a LeTourneau carry-all hauling 22 cubic yards on each trip. Figure 230 illustrates carry-all equipment in operation on airport construction.

Leveling and grading may be done by Bulldozer equipment such as shown in Figs. 231 through 234.

176. Compaction. The importance of good compaction of fill material has been stressed at various points throughout this book; one important piece of equipment used to accomplish this condition is the sheep's-foot tamping roller. Figure 235 illustrates several of these units in operation. These units are frequently used in combination with a disk harrow as shown in Fig. 236. The carry-all shown on the left has a capacity of 12 cubic yards.

Figure 237 shows a tractor-drawn sheep's-foot tamper compacting earth for airport runways.

An airport requires a large amount of trenching for the elaborate system of underdrain and sewer lines. Figure 238 shows a model 120 Buckeye trencher. The discharge conveyor can be shifted to discharge dirt to the left or right of the machine.

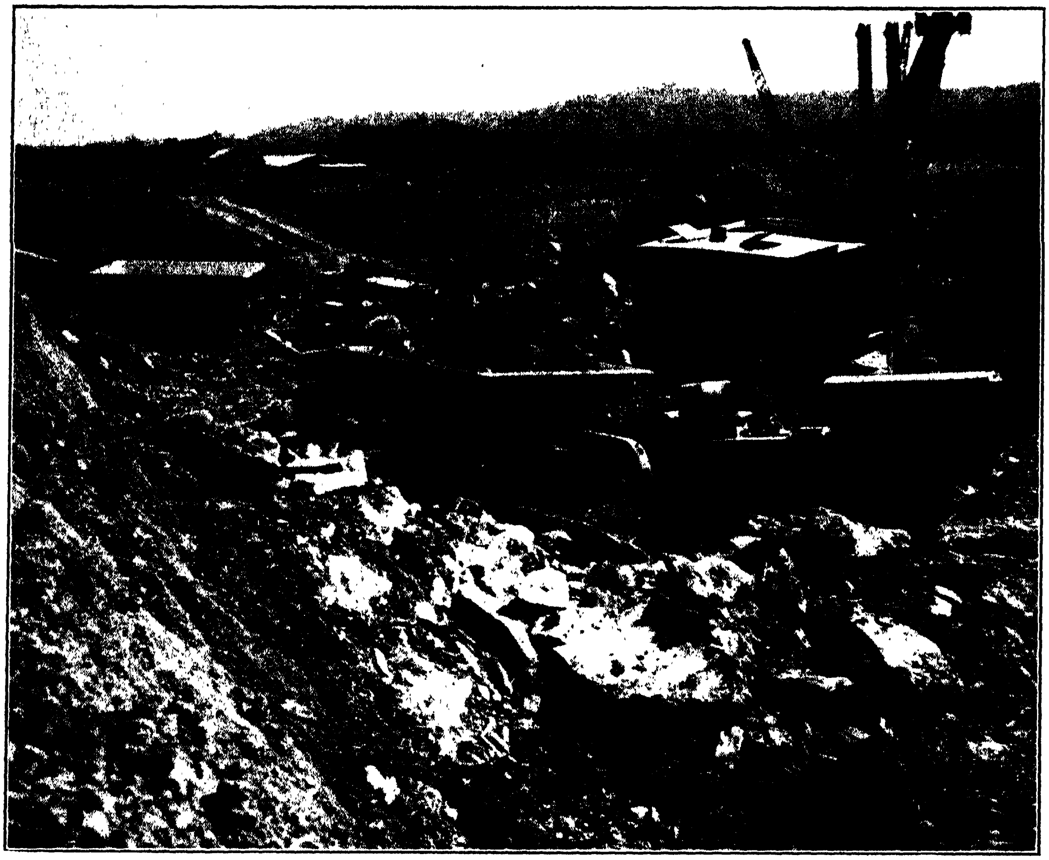
Figure 239 shows a model 410 Buckeye ladder-type trencher in use on airport drainage work in Canada. Some engineers call this a chain-bucket-type trencher.

Some drain pipe laid at airports is of relatively large size. The concrete pipe shown in Fig. 240 weighs 3½ tons per section. Each section is 8 feet long and has an inside diameter of 48 inches. A pipe of this size and weight requires special equipment for handling. This picture shows a caterpillar Diesel tractor with LeTourneau equipment for placing underdrains.

Figure 241 shows a line of tile underdrains being laid at an airport site. The equipment shown consists of a caterpillar Diesel tractor with a LeTourneau carry-all scraper being used to move the surplus dirt from the drainage ditch.

Figure 242 shows the equipment used in handling perforated steel pipe. A ditcher is operating on the right and a crane is shown on the left. Notice the batter boards spanning the ditch. An important element of construction is that underdrains be laid carefully to line and grade.

Many types of construction require mixing and pulverization. Figure 243 show a caterpillar Diesel tractor pulling a five-bottom plow mixing a stabilized base. The landing field in this case is to be a mixture of cement and desert sand.

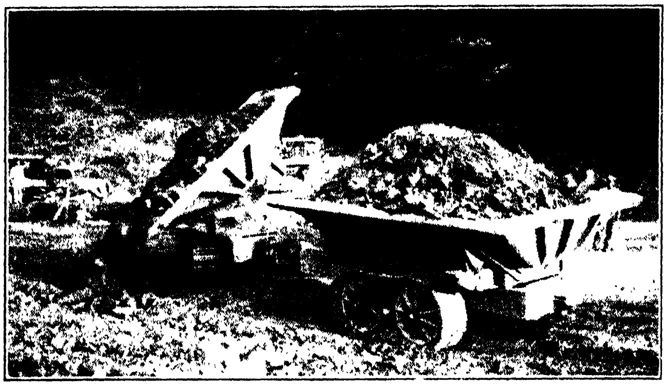


Courtesy Caterpillar Tractor Co.

Fig. 224. Heavy hauling equipment.

Courtesy Caterpillar Tractor Co.

Fig. 225. Side-dump wagons hauled in tandem.

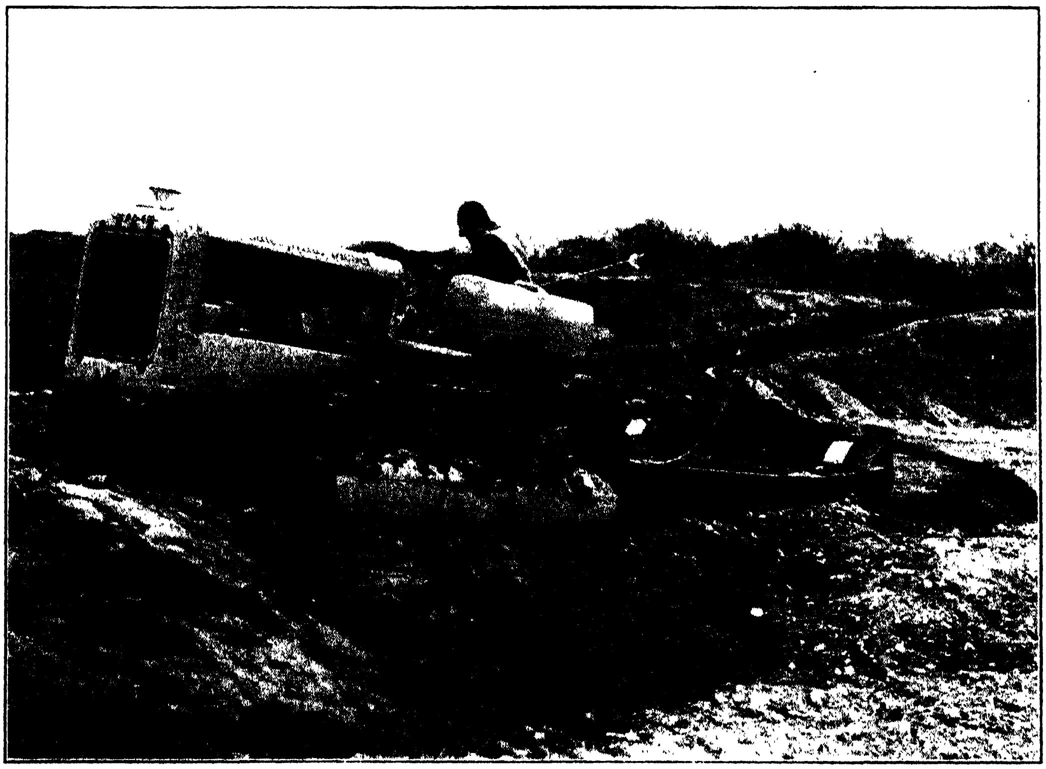


Courtesy Caterpillar Tractor Co.

FIG. 226. Unloading from side-dump wagons.

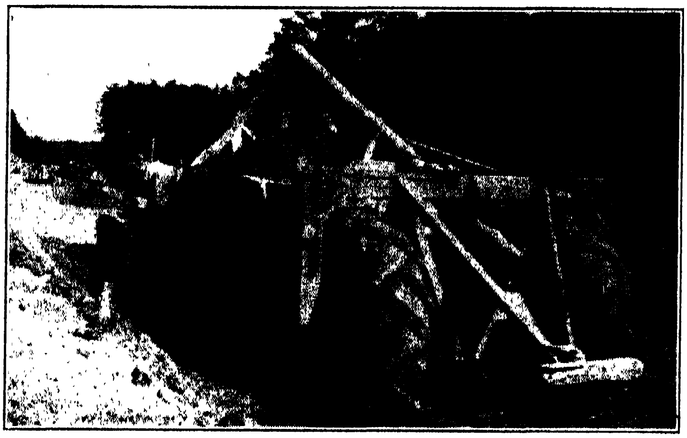
Courtesy Caterpillar Tractor Co.

FIG. 227. Breaking old pavement with a rooter.



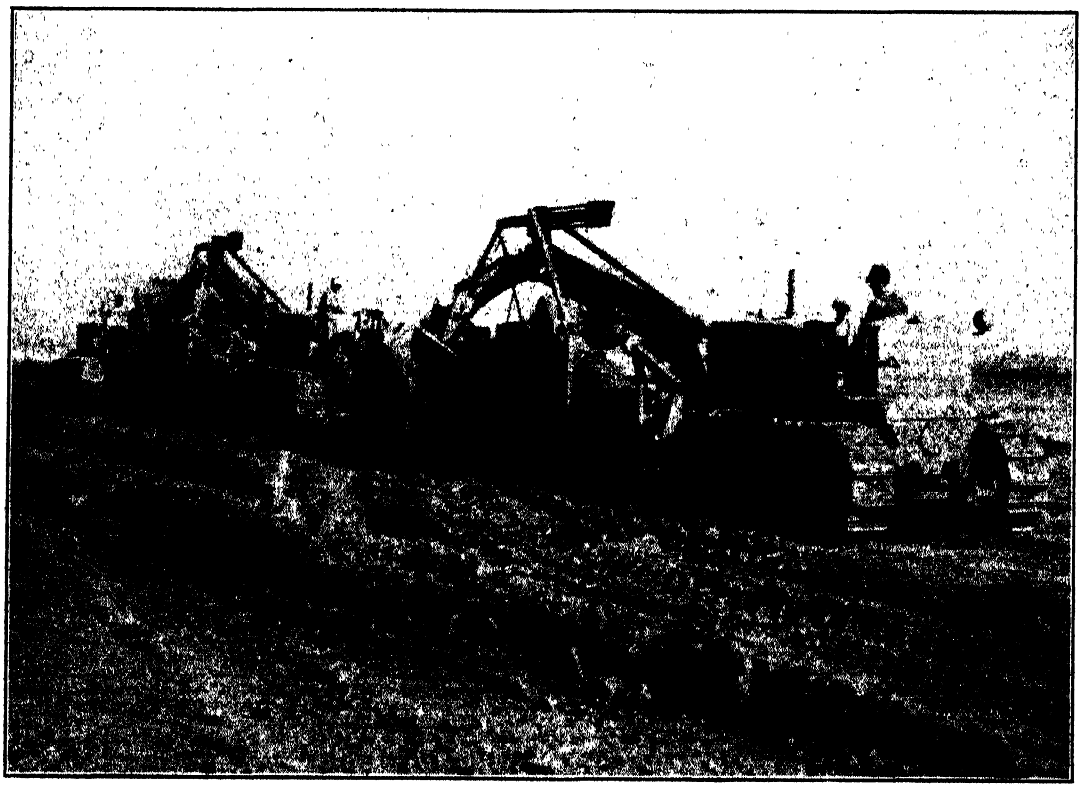
Courtesy Caterpillar Tractor Co.

FIG. 228. Scraper being used for short hauls.



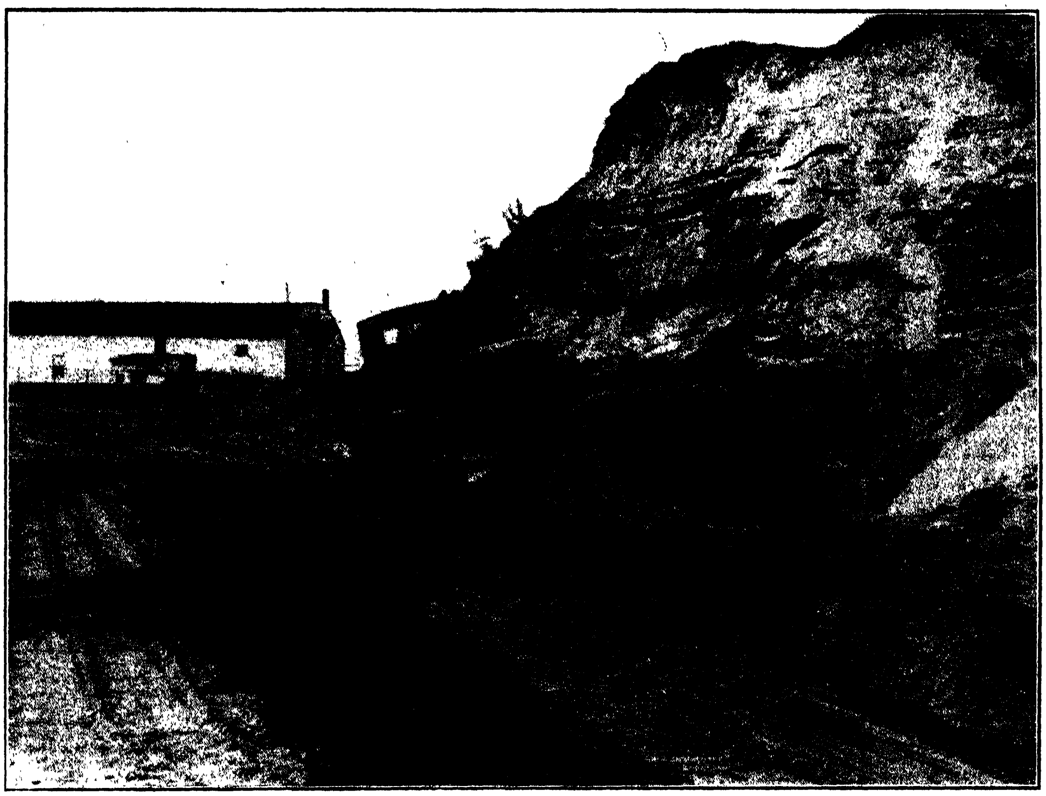
Courtesy Caterpillar Tractor Co.

FIG. 229. LeTourneau carry-all. capacity 22 cubic yards.



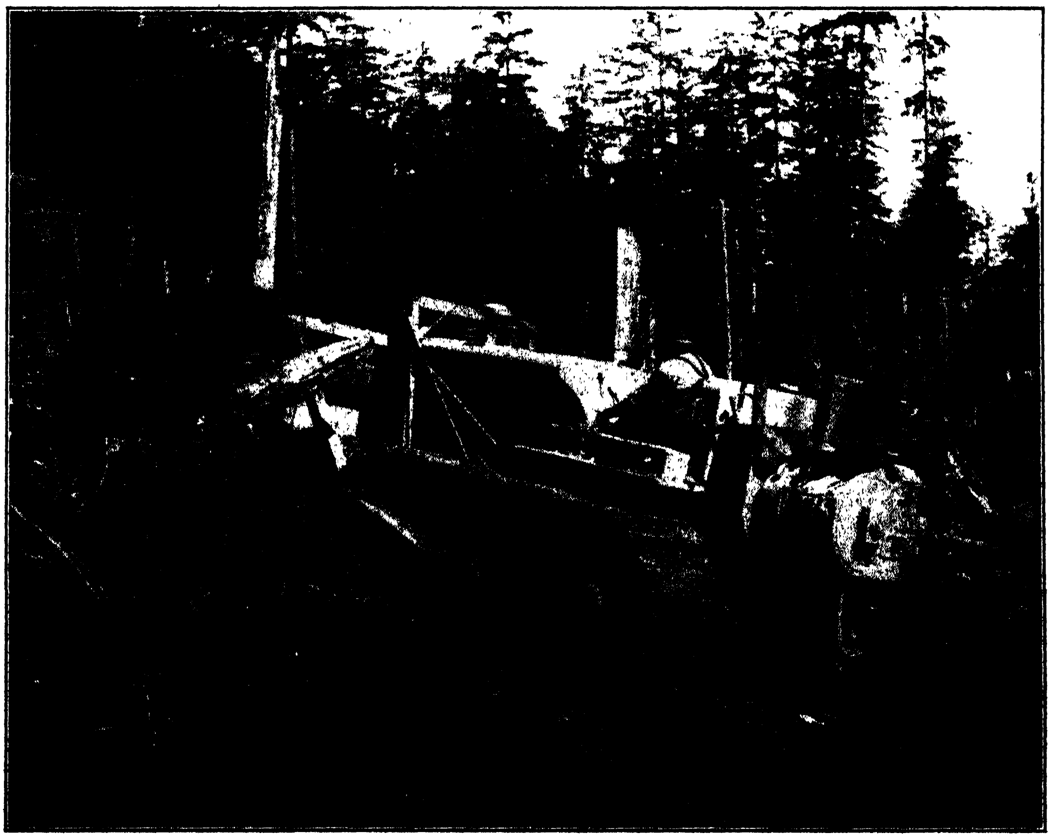
Courtesy Caterpillar Tractor Co.

FIG. 230. Carry-all equipment in operation.



Courtesy Caterpillar Tractor Co.

FIG. 231. Bulldozer in operation.

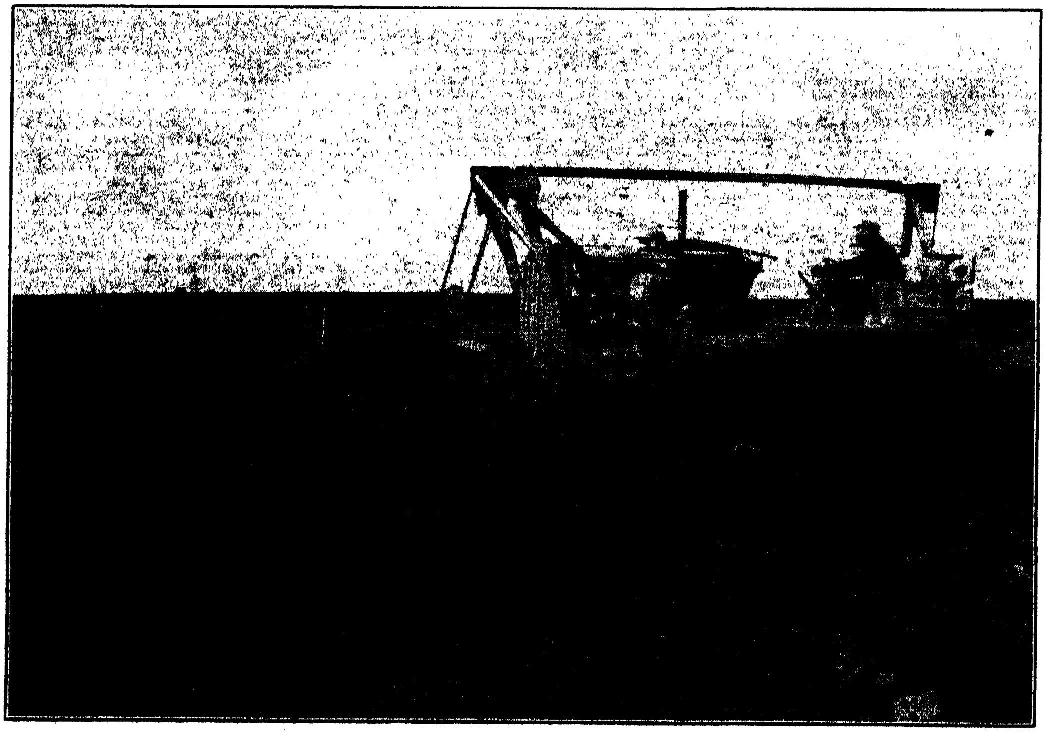


Courtesy Caterpillar Tractor Co.

FIG. 232. Bulldozer in operation.

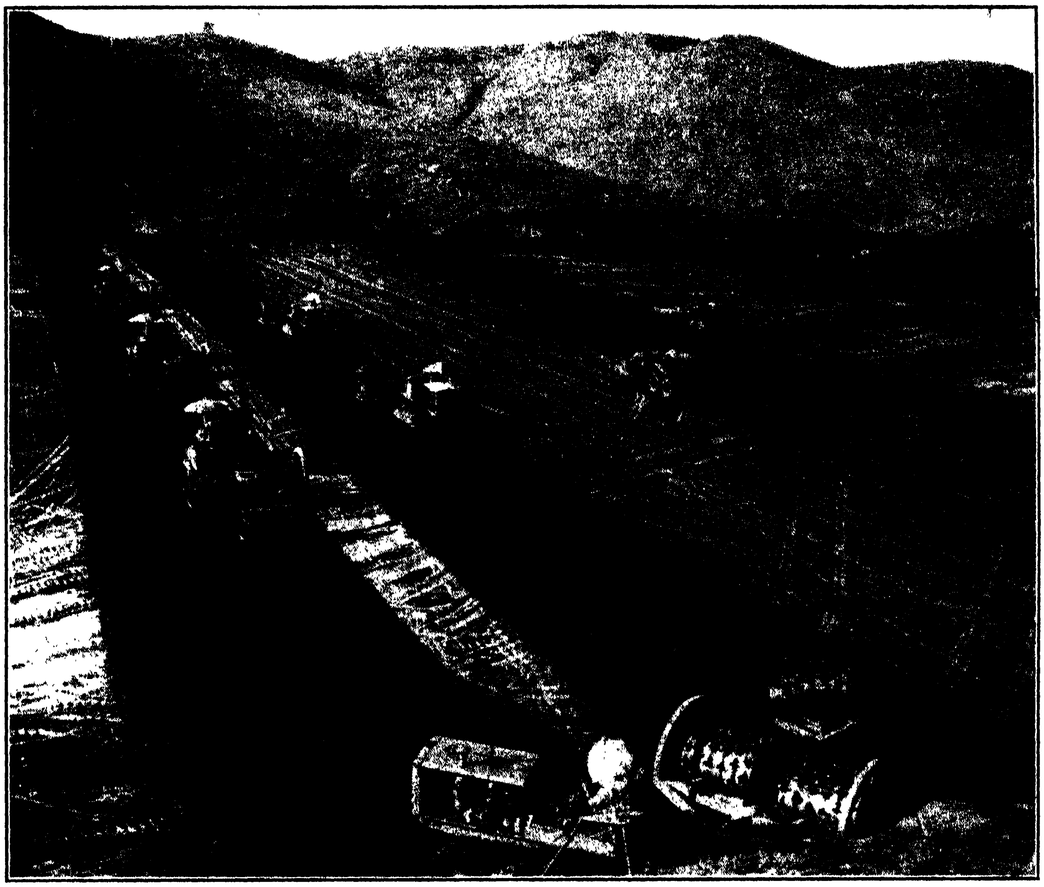
Courtesy Caterpillar Tractor Co.

FIG. 233. Bulldozer in operation.



Courtesy Caterpillar Tractor Co.

FIG. 234. Bulldozer in operation.

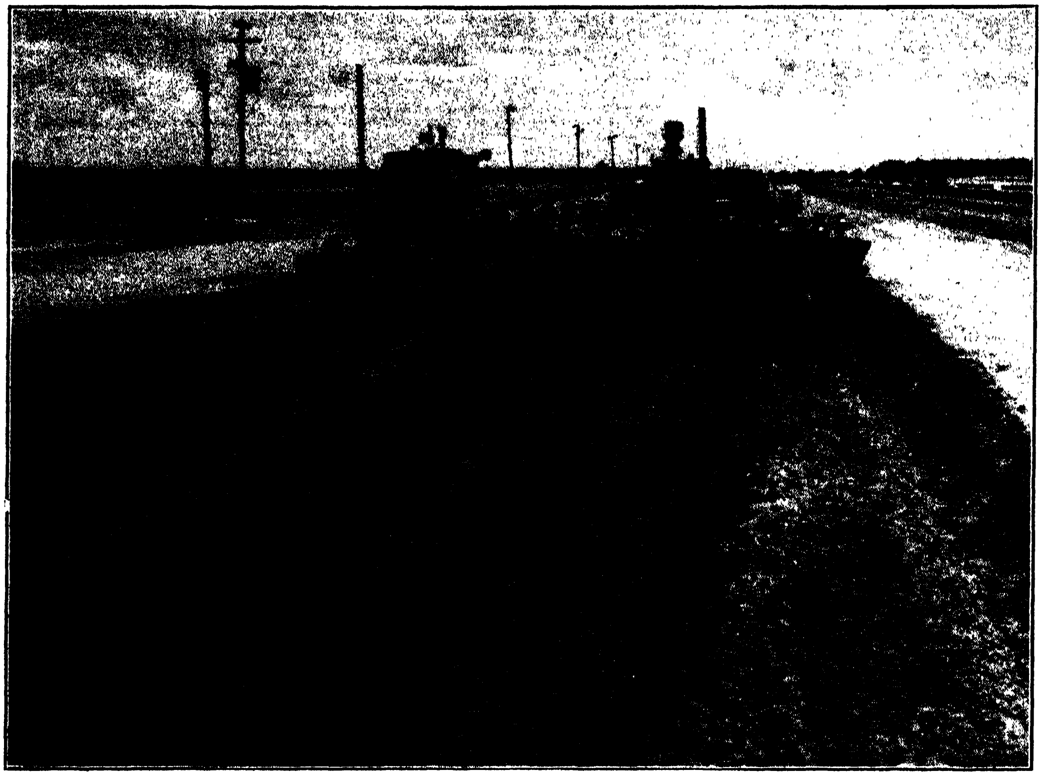


Courtesy Caterpillar Tractor Co.

FIG. 235. Compaction with sheep's-foot tampers.

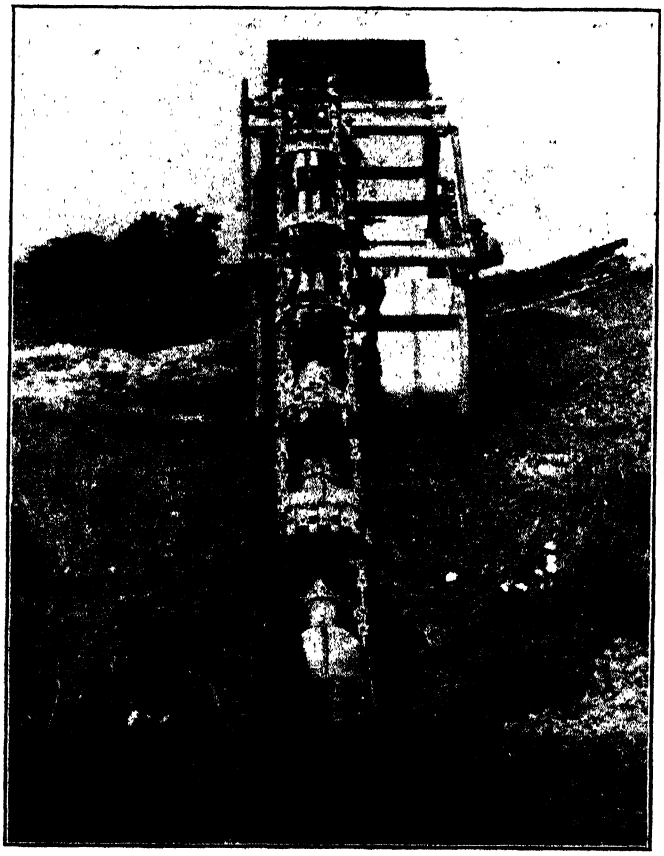
Courtesy Caterpillar Tractor Co.

FIG. 236. Compacting and mixing.



Courtesy Caterpillar Tractor Co.

FIG. 237. Spring-tooth harrow and sheep's-foot tamper in operation.

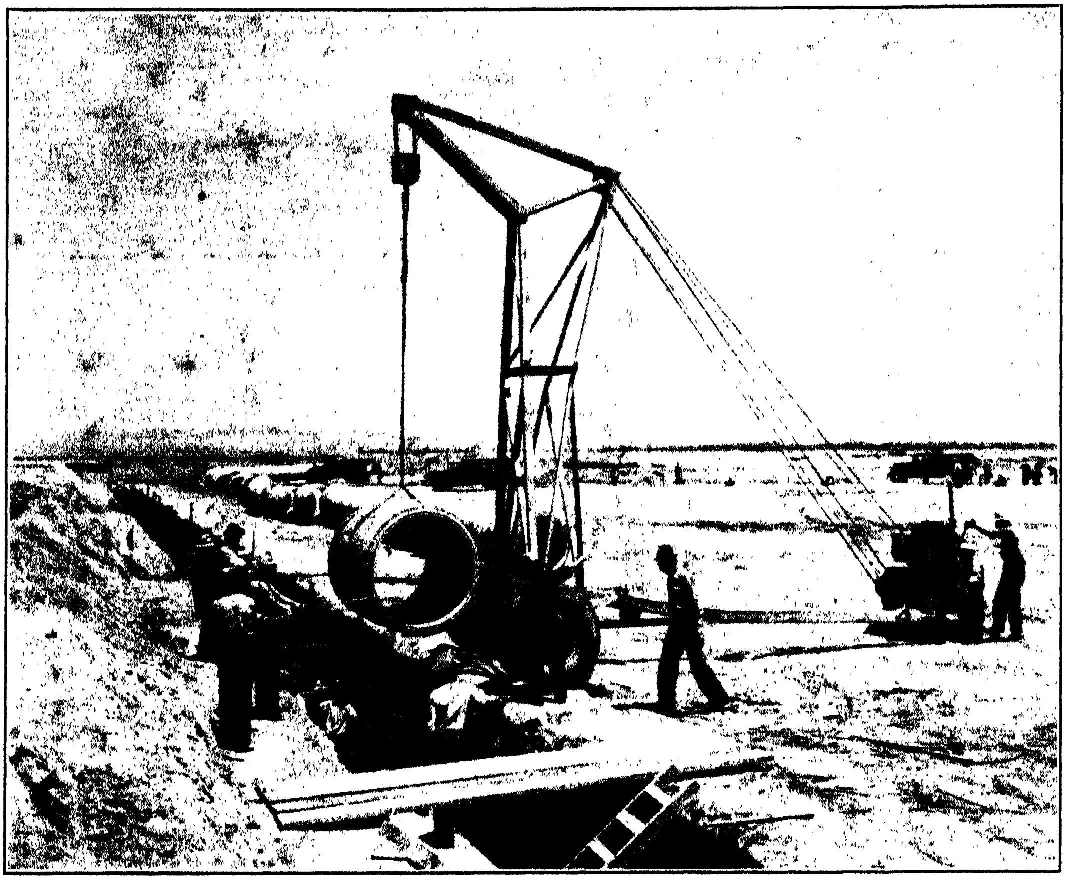


Courtesy The Buckeye Traction Ditcher Co.

FIG. 238. Model 120 Buckeye trencher.

Courtesy The Buckeye Traction Ditcher Co.

Fig. 239. Trencher operating to line and grade.

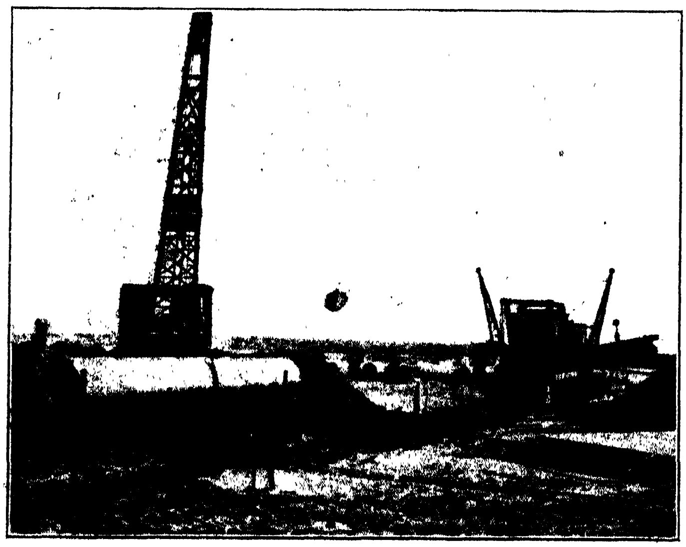


Courtesy Caterpillar Tractor Co.

FIG. 240. Large concrete pipe handled by crane.

Courtesy Caterpillar Tractor Co.

FIG. 241. Laying tile drain.



Courtesy Armco Drainage Products Association

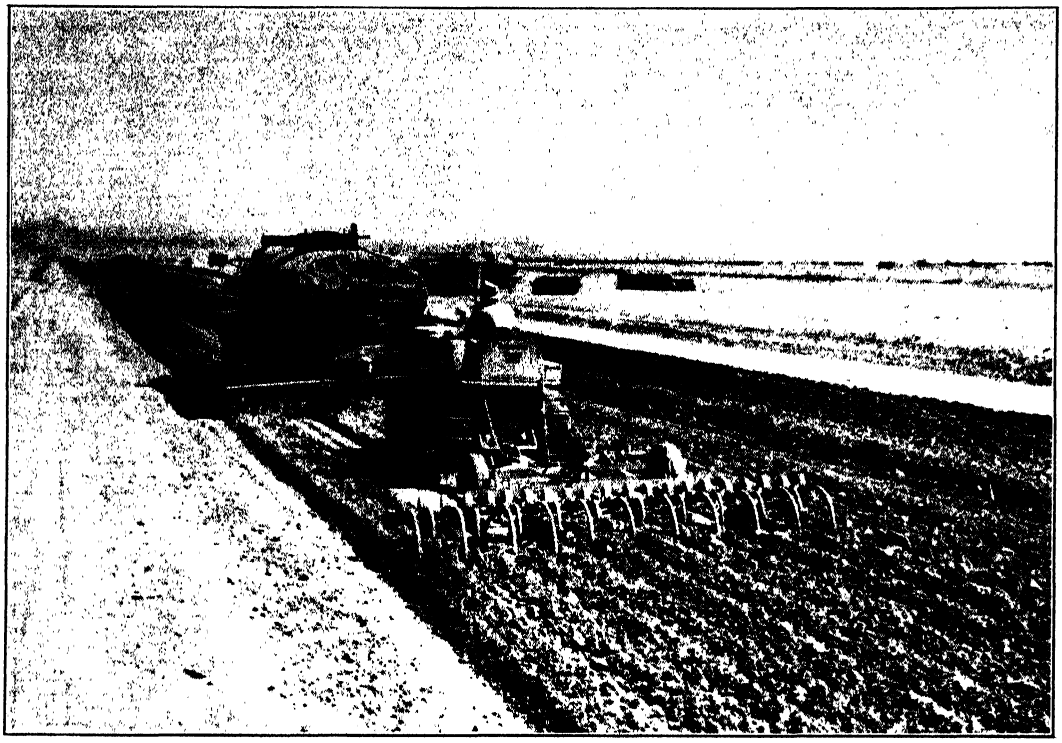
FIG. 242. Laying perforated steel pipe drain.

Courtesy Caterpillar Tractor Co.

FIG. 243. Five-bottom plow used in mixing stabilized base.

Courtesy Caterpillar Tractor Co.

FIG. 244. Pulverizing and mixing with disk harrow.



Courtesy Caterpillar Tractor Co.

FIG. 245. Pulverizing and mixing with spring-tooth harrow.

Figure 244 shows a caterpillar Diesel tractor with disk harrow disking a runway to pulverize and mix the subgrade material to a homogeneous condition.

Figure 245 shows a spring-tooth cultivator. In this operation the water is being applied to a mixture of sand and cement. The sprinkler is immediately followed by the cultivator which mixes the sand, cement, and water. The next operation is the leveling and rolling.

The edges of an airport must be cleaned up and all depressions eliminated. This is particularly important for a military airport, where emergency landings may occur. Figure 246 shows a fleet of caterpillar Diesel tractors with LeTourneau carry-all scrapers being used on general leveling and cleaning up.

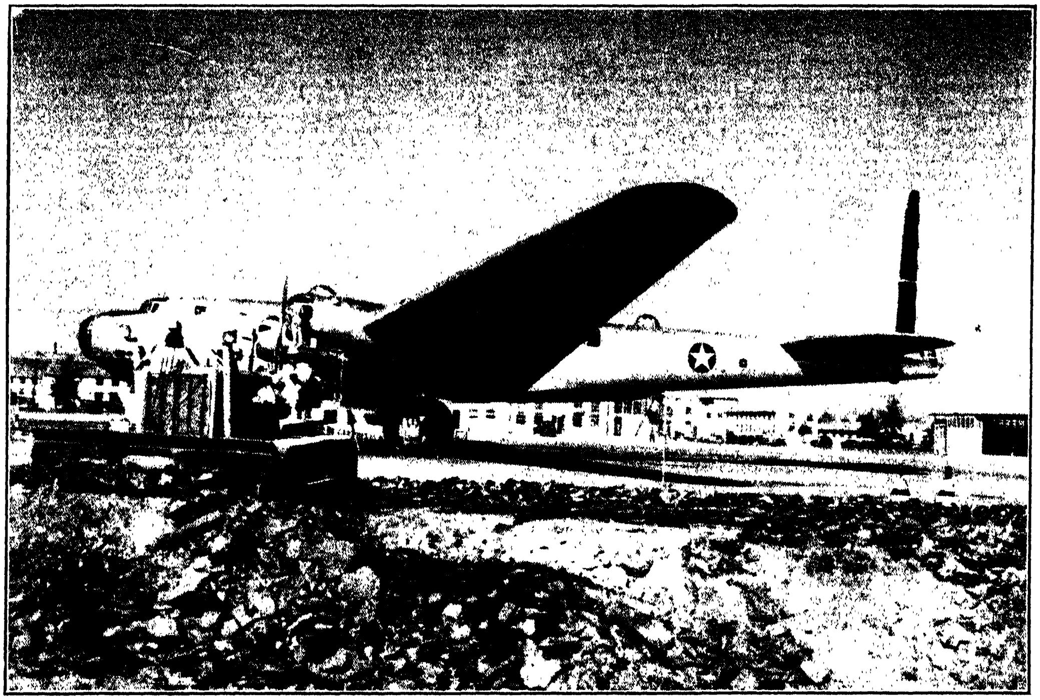
The maintenance of an airport depends to some extent upon the traffic. As heavier loads are applied to airport runways frequent resurfacing and repairs are necessary. Figure 247 shows a caterpillar Diesel bulldozer operating alongside of a B-19.

Figure 248 shows a Drott bull clam shovel. This shovel has a capacity of 1 cubic yard and the depth of cut is determined by the opening of the clam.

Snow presents a considerable problem at an airport. Sometimes the snow is removed and sometimes it is rolled down and used for a runway surface during the winter months. Figure 249 shows a load of snow in a 3-yard Drott bull clam snow shovel. Since the weight of snow is considerably less than dirt, the same size tractor can handle a larger bucket. When scraping up the snow, as when scraping dirt, the snow "boils up" into the bucket and compaction takes place, compacting the snow in a ratio of about 4 to 1. It is also possible for the operator to drive the unit up to a drift, take a bite, and transport the snow to the place of disposal. This same equipment may be used to load trucks. In this case a ramp must be provided. This method of removing snow is recommended in close places and places where snow cannot be pushed to one side and must be completely removed.

Courtesy Caterpillar Tractor Co.

FIG. 246. LeTourneau carry-all scrapers being used on general leveling and cleaning up.

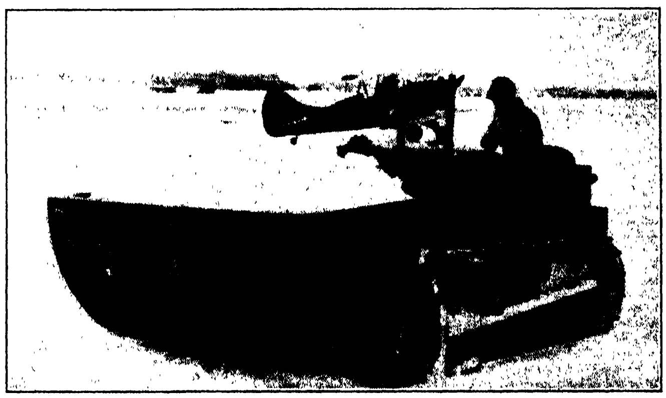


Courtesy Caterpillar Tractor Co.

FIG. 247. Bulldozer in operation alongside of a B-19.

Courtesy Hi-Way Service Corp.

FIG. 248. Drott bull clam shovel.



Courtesy Hi-Way Service Corp.

FIG. 249. Drott bull clam snow shovel.

Appendix

Diagrams for Determining the Discharge of Pipe Drains

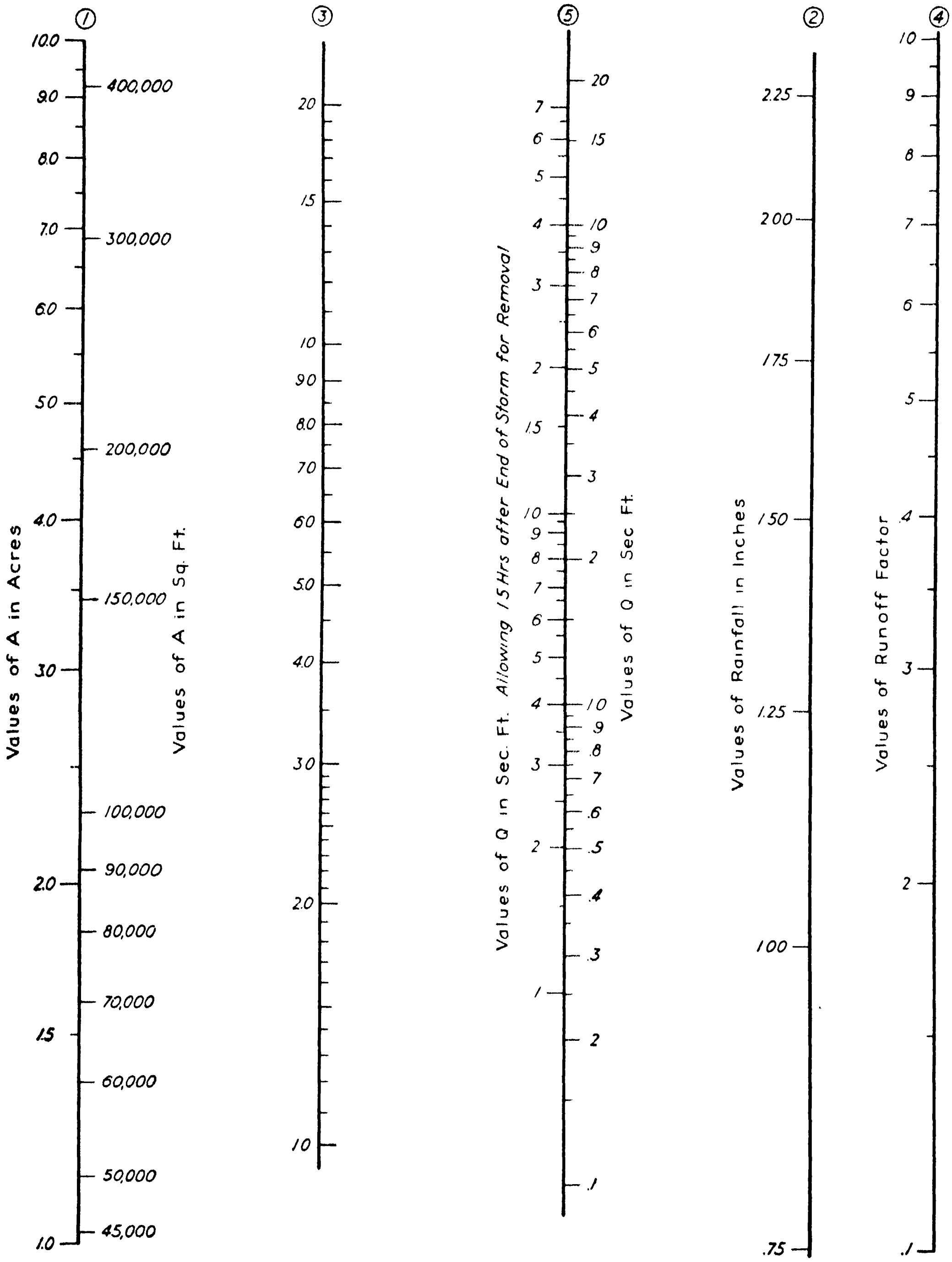


DIAGRAM 1.

To Use the Diagram.

If the area to be drained equals 100,000 sq. ft., rainfall equals 1.5 in., runoff factor equals 0.6, and, allowing 1.5 hours after end of storm for removal,

$$Q = \frac{100,000 \times 1.5 \times .6}{43,560(1 + 1.5)} = 0.826 \text{ sec.-ft.}$$

The solution by diagram is made as follows.

Connect value of A in square feet (100,000) on axis 1 with the value of rainfall in inches (1.5) on axis 2, secure the intersection on axis 3. Connect this point with the value of runoff factor (.6) on axis 4, and read answer (.83) on axis 5.

Diagram for runoff in second feet, $T = 1$ hour, $t = 1.5$ hours.

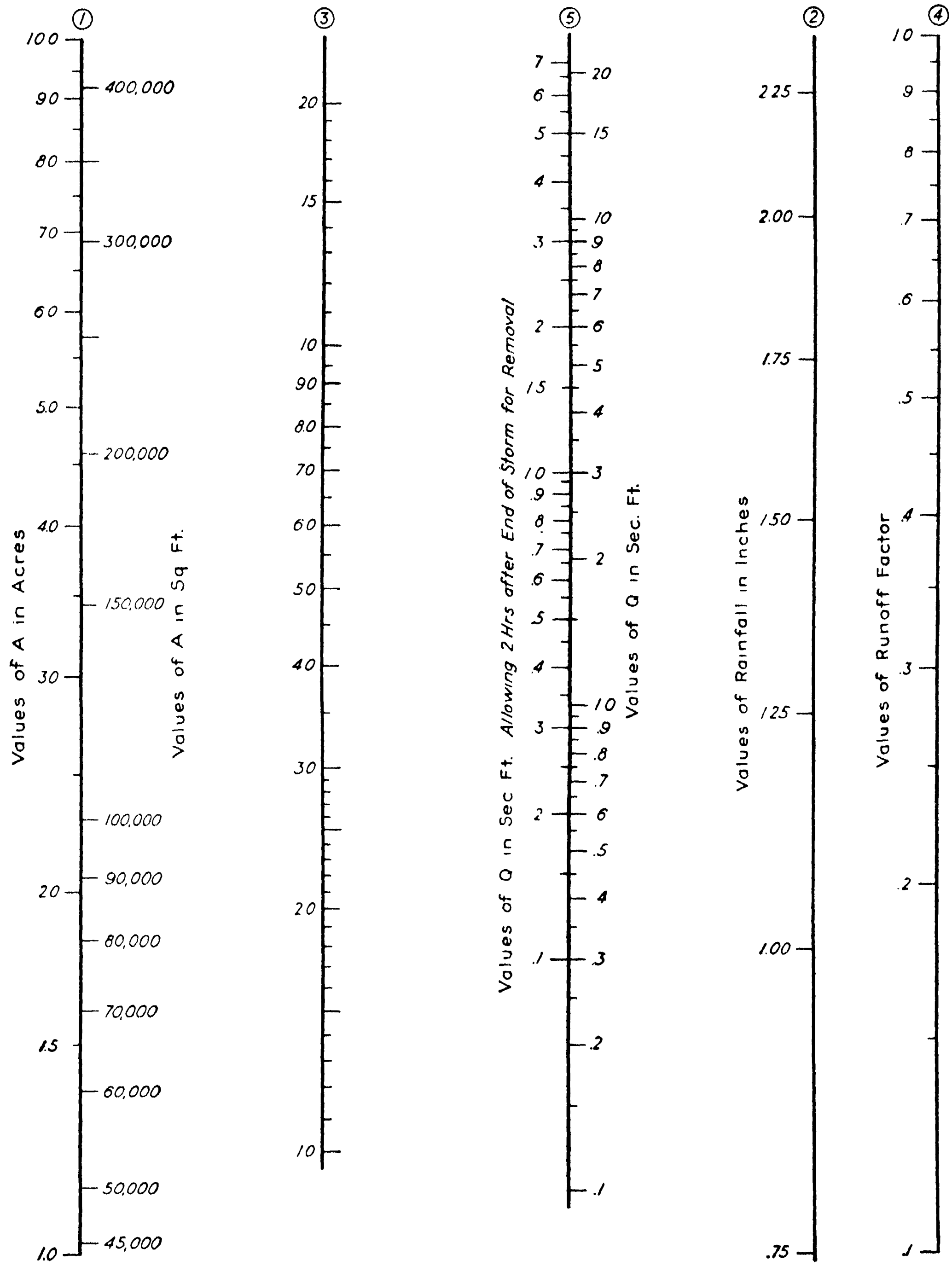


DIAGRAM 2.

To Use the Diagram.

If the area to be drained equals 100,000 sq. ft., rainfall equals 1.5 in., runoff factor equals 0.6, and, allowing 2 hours after end of storm for removal,

$$Q = \frac{100{,}000 \times 1.5 \times .6}{43{,}560(1 + 2)} = 0.687 \text{ sec.-ft.}$$

The solution by diagram is made as follows.

Connect value of A in square feet (100,000) on axis 1 with the value of rainfall in inches (1.5) on axis 2, secure the intersection on axis 3. Connect this point with the value of runoff factor (.6) on axis 4, and read answer (.69) on axis 5.

Diagram for runoff in second feet, $T = 1$ hour, $t = 2.0$ hours.

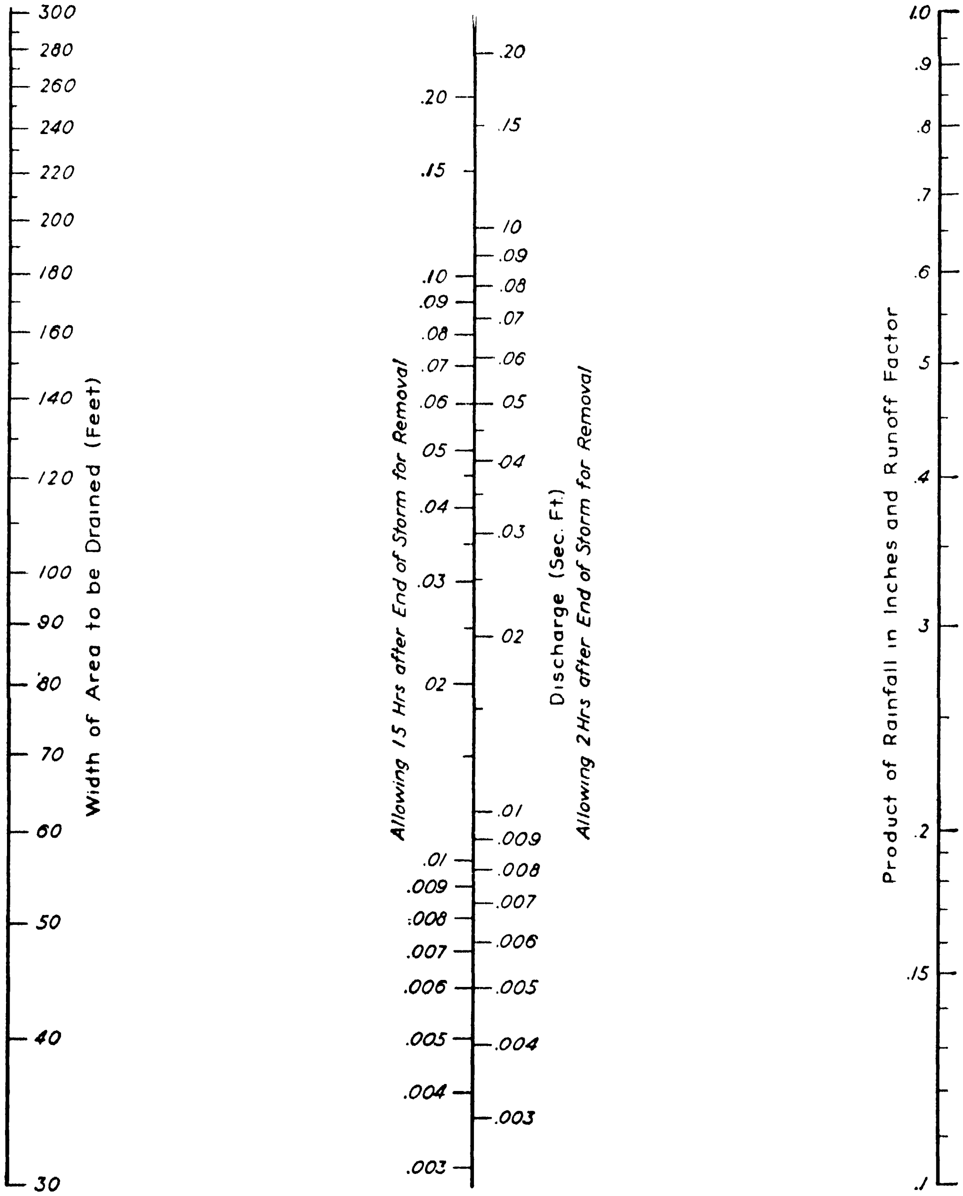


DIAGRAM 3.

To Use the Diagram.

Assume the width to be drained to be 100 ft., the rainfall to be 1.5 in. per hour and the runoff factor to be 0.4. Connect the value of width of area to be drained (100) with the value of the product of rainfall and runoff factor (1.5 × .4 = .6). Read off value of discharge per station on appropriate scale for allowable time after end of storm for removal. In this case if 1.5 hours are allowed, the discharge per station will be .055 sec.-ft.

Diagram for discharge per station in second feet.

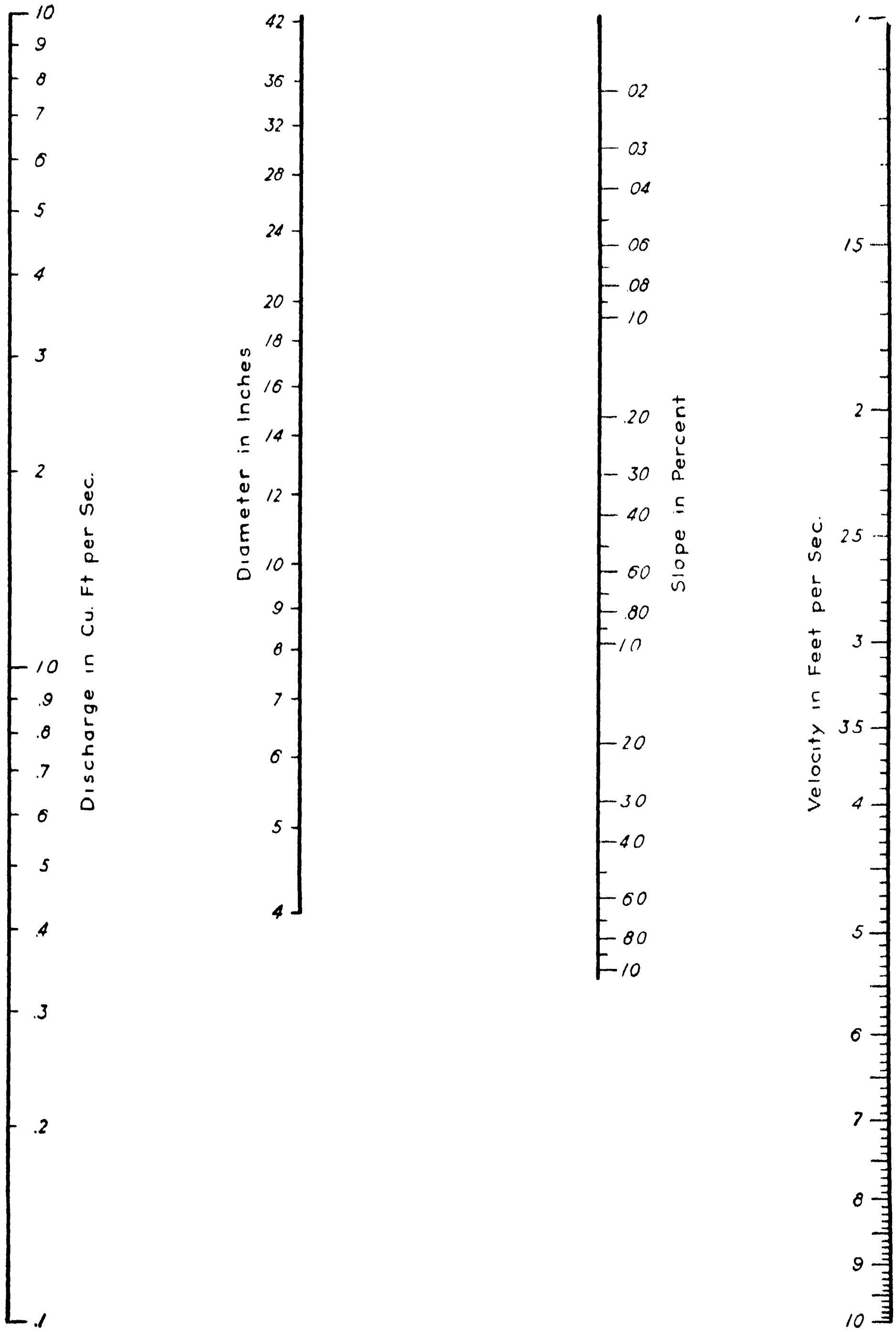


DIAGRAM 4.

To Use The Diagram.

Assume discharge to be 0.8 cu. ft. per sec. and slope of pipe to be 0.4 per cent. Connect value of discharge in cubic feet per second (.8) with the value of slope of pipe in per cent (.4) and read value of required diameters of pipe in inches (8.6). Use pipe of next larger size. Velocity in feet per second (2) may also be determined by extending line to where it crosses the velocity axis.

Diagram for finding size of pipe and velocity.

Index